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Eurocode 3 : Design of steel structures

Part 1.5 : Plated structural elements

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.5 :	Teil 1.5 :
Plaques planes	Aus Blechen zusammengesetzte Bauteile

Stage 34 draft

The technical improvements (doc. No. N1233E) agreed at the CEN/TC 250/SC 3 meeting in Madrid on 25 April 2003 and further editorial improvements are included in this version.

CEN

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National annex for EN 1993-1-5

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-1-5 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-5 through:

- 2.2(5)
- 3.3(1)
- 4.3(7)
- 5.1(2)
- 6.4(2)
- 8(2)
- 9.2.1(10)
- 10(1)
- C.2(1)
- C.5(2)
- C.8(1)
- C.9(5)

1 Introduction

1.1 Scope

(1) EN 1993-1-5 gives design requirements of stiffened and unstiffened plates which are subject to in-plane forces.

(2) These requirements are applicable to shear lag effects, effects of in-plane load introduction and effects from plate buckling for I-section plate girders and box girders. Plated structural components subject to inplane loads as in tanks and silos, are also covered. The effects of out-of-plane loading are not covered.

NOTE 1 The rules in this part complement the rules for class 1, 2, 3 and 4 sections, see EN 1993-1-1.

NOTE 2 For slender plates loaded with repeated direct stress and/or shear that are subjected to fatigue due to out of plane bending of plate elements (breathing) see EN 1993-2 and EN 1993-6.

NOTE 3 For the effects of out-of-plane loading and for the combination of in-plane effects and out-of-plane loading effects see EN 1993-2 and EN 1993-1-7.

NOTE 4 Single plate elements may be considered as flat where the curvature radius r satisfies:

$$r \geq \frac{b^2}{t} \tag{1.1}$$

where b is the panel width

t is the plate thickness

1.2 Normative references

(1) This European Standard incorporates, by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

EN 1993 Eurocode 3: Design of steel structures:

Part 1.1: General rules and rules for buildings;

1.3 Definitions

For the purpose of this standard, the following definitions apply:

1.3.1

elastic critical stress
stress in a component at which the component becomes unstable when using small deflection elastic theory of a perfect structure

1.3.2

membrane stress
stress at mid-plane of the plate

1.3.3

gross cross-section
the total cross-sectional area of a member but excluding discontinuous longitudinal stiffeners

1.3.4
effective cross-section (effective width)
the gross cross-section (width) reduced for the effects of plate buckling and/or shear lag; in order to distinguish between the effects of plate buckling, shear lag and the combination of plate buckling and shear lag the meaning of the word "effective" is clarified as follows:
"effective^{p}" for the effects of plate buckling
"effective^{s}" for the effects of shear lag
"effective" for the effects of plate buckling and shear lag

1.3.5
plated structure
a structure that is built up from nominally flat plates which are joined together; the plates may be stiffened or unstiffened

1.3.6
stiffener
a plate or section attached to a plate with the purpose of preventing buckling of the plate or reinforcing it against local loads; a stiffener is denoted:

- longitudinal if its direction is parallel to that of the member;
- transverse if its direction is perpendicular to that of the member.

1.3.7
stiffened plate
plate with transverse and/or longitudinal stiffeners

1.3.8
subpanel
unstiffened plate portion surrounded by flanges and/or stiffeners

1.3.9
hybrid girder
girder with flanges and web made of different steel grades; this standard assumes higher steel grade in flanges

1.3.10
sign convention
unless otherwise stated compression is taken as positive

1.4 Symbols

(1) In addition to those given in EN 1990 and EN 1993-1-1, the following symbols are used:

$A_{s\ell}$ total area of all the longitudinal stiffeners of a stiffened plate;

A_{st} gross cross sectional area of one transverse stiffener;

A_{eff} effective cross sectional area;

$A_{c,eff}$ effective^{p} cross sectional area;

$A_{c,eff,loc}$ effective^{p} cross sectional area for local buckling;

a length of a stiffened or unstiffened plate;

b width of a stiffened or unstiffened plate;

b_w clear width between welds;

b_{eff} effective^{s} width for elastic shear lag;

F_{Ed} design transverse force;

h_w clear web depth between flanges;

L_{eff} effective length for resistance to transverse forces, see 6;

$M_{f.Rd}$ design plastic moment of resistance of a cross-section consisting of the flanges only;

$M_{pl.Rd}$ design plastic moment of resistance of the cross-section (irrespective of cross-section class);

M_{Ed} design bending moment;

N_{Ed} design axial force;

t thickness of the plate;

V_{Ed} design shear force including shear from torque;

W_{eff} effective elastic section modulus;

β effective^s width factor for elastic shear lag;

(2) Additional symbols are defined where they first occur.

2 Basis of design and modelling

2.1 General

(1)P The effects of shear lag and plate buckling shall be taken into account if these significantly influence the structural behaviour at the ultimate, serviceability or fatigue limit states.

2.2 Effective width models for global analysis

(1)P The effects of shear lag and of plate buckling on the stiffness of members and joints shall be taken into account if this significantly influences the global analysis.

(2) The effects of shear lag of flanges in elastic global analysis may be taken into account by the use of an effective^s width. For simplicity this effective^s width may be assumed to be uniform over the length of the beam.

(3) For each span of a beam the effective^s width of flanges should be taken as the lesser of the full width and L/8 per side of the web, where L is the span or twice the distance from the support to the end of a cantilever.

(4) The effects of plate buckling in elastic global analysis may be taken into account by effective^p cross sectional areas of the elements in compression, see 4.3.

(5) For global analysis the effect of plate buckling on the stiffness may be ignored when the effective^p cross-sectional area of an element in compression is larger than ρ_{lim} times the gross cross-sectional area.

NOTE The parameter ρ_{lim} may be determined in the National Annex. The value ρ_{lim} = 0,5 is recommended. If this condition is not fulfilled a reduced stiffness according to 7.1 of EN 1993-1-3 may be used.

2.3 Plate buckling effects on uniform members

(1) Effective^p width models for direct stresses, resistance models for shear buckling and buckling due to transverse loads as well as interactions between these models for determining the resistance of uniform members at the ultimate limit state may be used when the following conditions apply:

- panels are rectangular and flanges are parallel within an angle not greater than α_{limit} = 10°
- an open hole or cut out is small and limited to a diameter d that satisfies d/h ≤ 0,05, where h is the width of the plate

NOTE 1 Rules are given in section 4 to 7.

NOTE 2 For angles greater than α_{limit} non-rectangular panels may be checked assuming a fictional rectangular panel based on the largest dimensions a and b of the panel.

(2) For the calculation of stresses at the serviceability and fatigue limit state the effective^{s} area may be used if the condition in 2.2(5) is fulfilled. For ultimate limit states the effective area according to 3.3 should be used with β replaced by β_{ult}.

2.4 Reduced stress method

(1) As an alternative to the use of the effective^{p} width models for direct stresses given in sections 4 to 7, the cross sections may be assumed to be class 3 sections provided that the stresses in each panel do not exceed the limits specified in section 10.

NOTE The reduced stress method is equivalent to the effective^{p} width method (see 2.3) for single plated elements. However, in verifying the stress limitations no load shedding between plated elements of a cross section is accounted for.

2.5 Non uniform members

(1) Methods for non uniform members (e.g. with haunched beams, non rectangular panels) or with regular or irregular large openings may be based on FE-calculations.

NOTE 1 Rules are given in Annex B.

NOTE 2 For FE-calculations see Annex C.

2.6 Members with corrugated webs

(1) In the analysis of structures with members with corrugated webs, the bending stiffness may be based on the contributions of the flanges only and webs may be considered to transfer shear and transverse loads only.

NOTE For plate buckling resistance of flanges in compression and the shear resistance of webs see Annex D.

3 Shear lag effects in member design

3.1 General

(1) Shear lag in flanges may be neglected provided that $b_0 < L_e/50$ where the flange width b_0 is taken as the outstand or half the width of an internal element and L_e is the length between points of zero bending moment, see 3.2.1(2).

NOTE At ultimate limit state, shear lag in flanges may be neglected if $b_0 < L_e/20$.

(2) Where the above limit is exceeded the effect of shear lag in flanges should be considered at serviceability and fatigue limit state verifications by the use of an effective^{s} width according to 3.2.1 and a stress distribution according to 3.2.2. For ultimate limit states an effective width according to 3.3 may be used.

(3) Stresses under elastic conditions from the introduction of in-plane local loads into the web through a flange should be determined from 3.2.3.

3.2 Effective[s] width for elastic shear lag

3.2.1 Effective width factor for shear lag

(1) The effective[s] width b_{eff} for shear lag under elastic conditions should be determined from:

$$b_{eff} = \beta \, b_0 \tag{3.1}$$

where the effective[s] factor β is given in Table 3.1.

This effective width may be relevant for serviceability and fatigue limit states.

(2) Provided adjacent internal spans do not differ more than 50% and any cantilever span is not larger than half the adjacent span the effective lengths L_e may be determined from Figure 3.1. In other cases L_e should be taken as the distance between adjacent points of zero bending moment.

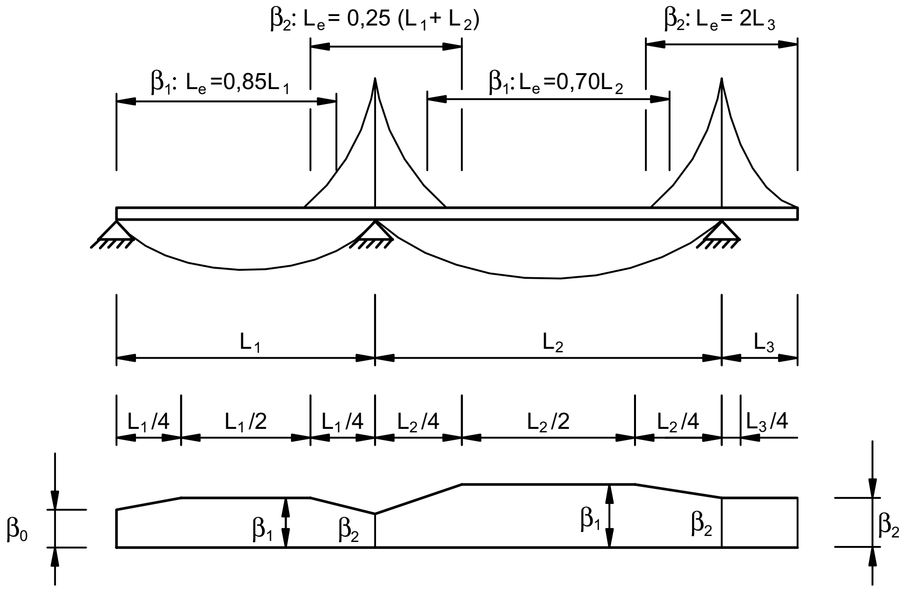
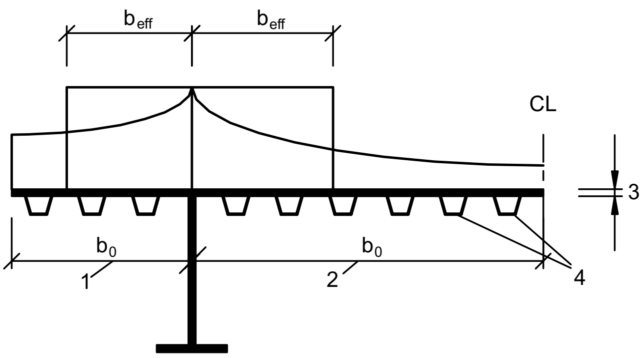


Figure 3.1: Effective length L_e for continuous beam and distribution of effective[s] width



1 for outstand flange
2 for internal flange
3 plate thickness t
4 stiffeners with $A_{s\ell} = \sum A_{s\ell i}$

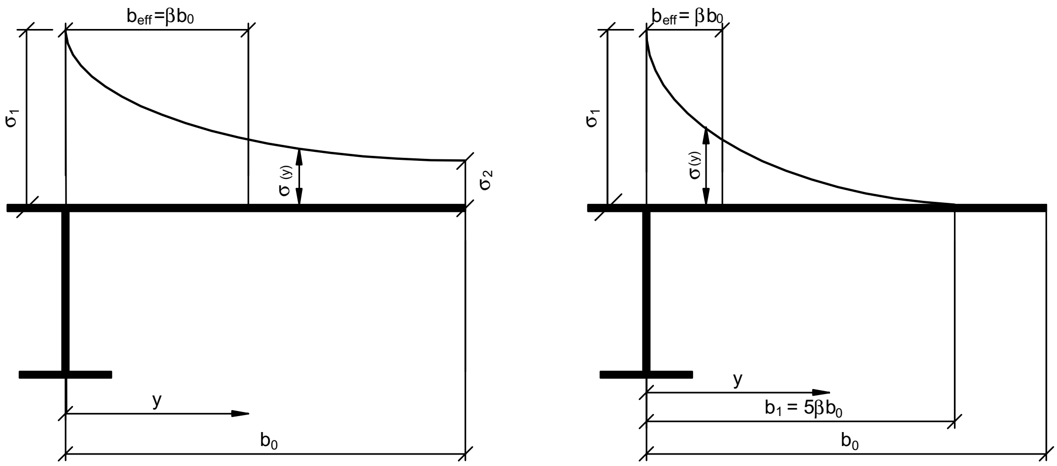
Figure 3.2: Definitions of notation for shear lag

Table 3.1: Effective^{s} width factor β

κ	location for verification	β – value
$\kappa \quad 0,02$		$\beta = 1,0$
$0,02 < \kappa \quad 0,70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{1 + 6,4\,\kappa^2}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{1 + 6,0\left(\kappa - \dfrac{1}{2500\,\kappa}\right) + 1,6\,\kappa^2}$
$> 0,70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{5,9\,\kappa}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{8,6\,\kappa}$
all κ	end support	$\beta_0 = (0,55 + 0,025 / \kappa)\,\beta_1$, but $\beta_0 < \beta_1$
all κ	cantilever	$\beta = \beta_2$ at support and at the end
$\kappa = \alpha_0\, b_0 / L_e$ with $\alpha_0 = \sqrt{1 + \dfrac{A_{s\ell}}{b_0 t}}$ in which $A_{s\ell}$ is the area of all longitudinal stiffeners within the width b_0 and other symbols are as defined in Figure 3.1 and Figure 3.2.		

3.2.2 Stress distribution for shear lag

(1) The distribution of longitudinal stresses across the plate due to shear lag should be obtained from Figure 3.3.



$\beta > 0,20:$

$\sigma_2 = 1,25\,(\beta - 0,20)\,\sigma_1$

$\sigma(y) = \sigma_2 + (\sigma_1 - \sigma_2)\,(1 - y / b_0)^4$

$\beta < 0,20:$

$\sigma_2 = 0$

$\sigma(y) = \sigma_1 (1 - y / b_1)^4$

σ_1 is calculated with the effective width of the flange b_{eff}

Figure 3.3: Distribution of stresses across the plate due to shear lag

3.2.3 In-plane load effects

(1) The elastic stress distribution in a stiffened or unstiffened plate due to the local introduction of in-plane forces (see Figure 3.4) should be determined from:

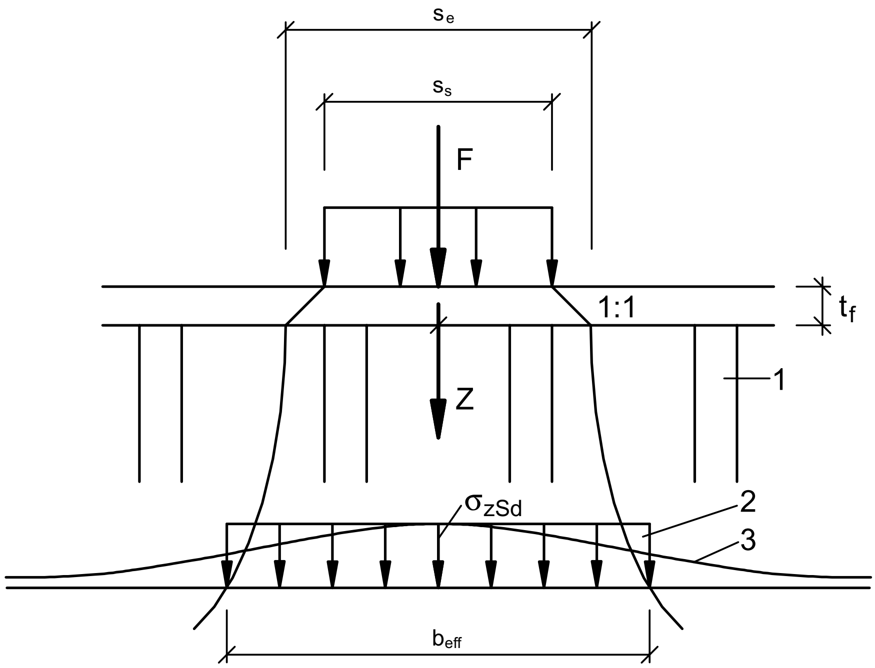
$$\sigma_{z,Ed} = \frac{F_{Sd}}{b_{eff}\left(t + a_{st,1}\right)} \tag{3.2}$$

with: $b_{eff} = s_e \sqrt{1 + \left(\frac{z}{s_e\ n}\right)^2}$

$$n = 0{,}636 \sqrt{1 + \frac{0{,}878\ a_{st,1}}{t}}$$

$$s_e = s_s + 2\ t_f$$

where $a_{st,1}$ is the gross cross-sectional area of the smeared stiffeners per unit length, i.e. the area of the stiffener divided by the centre to centre distance;



1 stiffener
2 simplified stress distribution
3 actual stress distribution

Figure 3.4: In-plane load introduction

NOTE The stress distribution may be relevant for the fatigue verification.

3.3 Shear lag at ultimate limit states

(1) At ultimate limit states shear lag effects may be determined using one of the following methods:

a) elastic shear lag effects as defined for serviceability and fatigue limit states,

b) interaction of shear lag effects with geometric effects of plate buckling,

c) elastic-plastic shear lag effects allowing for limited plastic strains.

NOTE 1 The National Annex may choose the method to be applied.

NOTE 2 The geometric effects of plate buckling on shear lag may be taken into account by using A_{eff} given by

$$A_{eff} = A_{c,eff}\beta_{ult} \tag{3.3}$$

where $A_{c,eff}$ is the effective[p] area for a compression flange with respect to plate buckling from 4.4 and 4.5

β_{ult} is the effective[s] width factor for the effect of shear lag at the ultimate limit state, which may be taken as β determined from Table 3.1 with α_0 replaced by

$$\alpha_0^* = \sqrt{\frac{A_{c,eff}}{b_0\ t}} \tag{3.4}$$

NOTE 3 Elastic-plastic shear lag effects allowing for limited plastic strains may be taken into account by using A_{eff} given by

$$A_{eff} = A_{c,eff}\beta^{\kappa} \geq A_{c,eff}\beta \tag{3.5}$$

where β and κ are calculated from Table 3.1.

The expression in NOTE 2 and NOTE 3 may also be applied for flanges in tension in which case $A_{c,eff}$ should be replaced by the gross area of the tension flange.

4 Plate buckling effects due to direct stresses

4.1 General

(1) This section gives rules to account for plate buckling effects from direct stresses at the ultimate limit state when the following criteria are met:

a) The panels are rectangular and flanges are parallel within the angle limit stated in 2.3.

b) Stiffeners if any are provided in the longitudinal and/or transverse direction.

c) Open holes or cut outs are small (see 2.3).

d) Members are of uniform cross section.

e) No flange induced web buckling occurs.

NOTE 1 For requirements to prevent compression flange buckling in the plane of the web see section 8.

NOTE 2 For stiffeners and detailing of plated members subject to plate buckling see section 9.

4.2 Resistance to direct stresses

(1) The resistance of plated members to direct stresses may be determined using effective[p] areas of plate elements in compression for calculating class 4 cross sectional data (A_{eff}, I_{eff}, W_{eff}) to be used for cross sectional verifications or for member verifications for column buckling or lateral torsional buckling according to EN 1993-1-1.

> **NOTE 1** In this method load shedding between various plate elements is implicitly taken into account.
>
> **NOTE 2** For member verifications see EN 1993-1-1.

(2) Effective[p] areas may be determined on the basis of initial linear strain distributions resulting from elementary bending theory under the reservations of applying 4.4(5) and (6). These distributions are limited by the attainment of yield strain in the mid plane of the compression flange plate.

> **NOTE** Excessive strains in the tension zone are controlled by the yield strain limit in the compression zone and the remaining parts of the cross section.

4.3 Effective cross section

(1) In calculating design longitudinal stresses, account should be taken of the combined effect of shear lag and plate buckling using the effective areas given in 3.3.

(2) The effective cross section properties of members should be based on the effective areas of the compression elements and on the effective[s] area of the tension elements due to shear lag, and their locations within the effective cross section.

(3) The effective area A_{eff} should be determined assuming the cross section is subject only to stresses due to uniform axial compression. For non-symmetrical cross sections the possible shift e_N of the centroid of the effective area A_{eff} relative to the centre of gravity of the gross cross-section, see Figure 4.1, gives an additional moment which should be taken into account in the cross section verification using 4.6.

(4) The effective section modulus W_{eff} should be determined assuming the cross section is subject only to bending stresses, see Figure 4.2. For biaxial bending effective section moduli should be determined for both main axes.

(5) As an alternative to 4.3(3) and (4) a single effective section may be determined for the resulting state of stress from compression and bending acting simultaneously. The effects of e_N should be taken into account as in 4.3(3). This requires an iterative procedure.

(6) The stress in a flange should be calculated using the elastic section modulus with reference to the mid-plane of the flange.

(7) Hybrid girders may have flange material with yield strength f_{yf} up to 1 to $\varphi_h \times f_{yw}$ provided that:

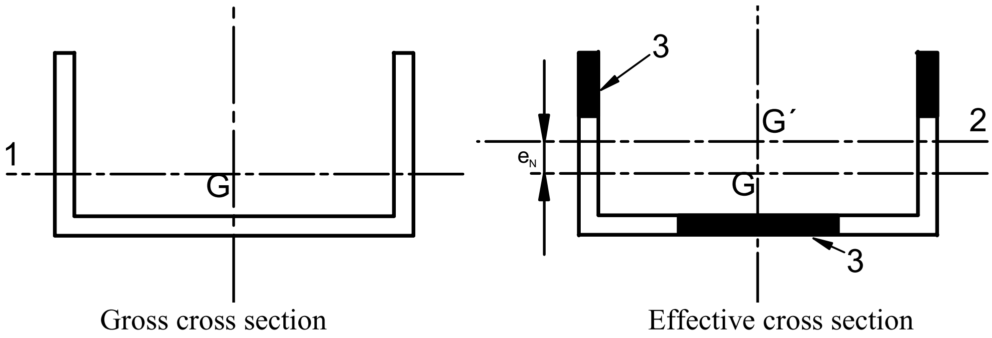
a) the increase of flange stresses caused by yielding of the web is taken into account by limiting the stresses in the web to f_{yw}

b) f_{yf} (rather than f_{yw}) is used in determining the effective area of the web.

> **NOTE** The National annex may specify the value φ_h. A value of $\varphi_h = 2,0$ is recommended.

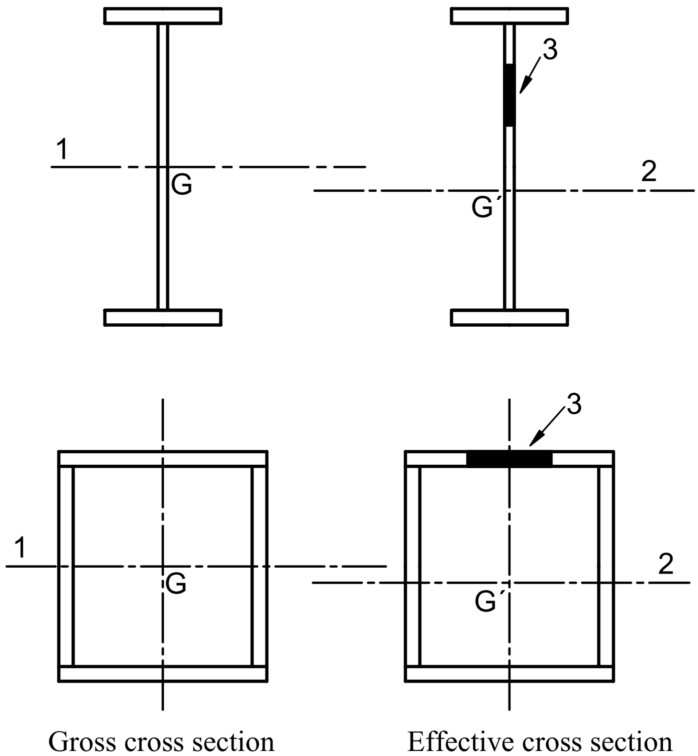
(8) The increase of deformations and of stresses at serviceability and fatigue limit states may be ignored for hybrid girders complying with 4.3(7).

(9) For hybrid girders complying with 4.3(7) the stress range limit in EN 1993-1-9 may be taken as $1,5f_{yf}$.



G centroid of the gross (fully effective) cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Figure 4.1: Class 4 cross-sections - axial force



G centroid of the gross (fully effective) cross section
G´ centroid of the effective cross section
1 centroidal axis of the gross cross section
2 centroidal axis of the effective cross section
3 non effective zone

Figure 4.2: Class 4 cross-sections - bending moment

4.4 Plate elements without longitudinal stiffeners

(1) The effective[p] areas of flat compression elements should be obtained using Table 4.1 for internal elements and Table 4.2 for outstand elements. The effective[p] area of the compression zone of a plate with the gross cross-sectional area A_c should be obtained from:

$$A_{c,eff} = \rho \, A_c \tag{4.1}$$

where ρ is the reduction factor for plate buckling.

(2) The reduction factor ρ may be taken as follows:

– internal compression elements:

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \le 1{,}0 \tag{4.2}$$

– outstand compression elements:

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \le 1{,}0 \tag{4.3}$$

with $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\bar{b}$ is the appropriate width as follows (for definitions, see Table 5.2 of EN 1993-1-1)

- b_w for webs;
- b for internal flange elements (except RHS);
- $b - 3\,t$ for flanges of RHS;
- c for outstand flanges;
- h for equal-leg angles;
- h for unequal-leg angles;

k_σ is the buckling factor corresponding to the stress ratio ψ and boundary conditions. For long plates k_σ is given in Table 4.1 or Table 4.2 as appropriate;

t is the thickness;

σ_{cr} is the elastic critical plate buckling stress see Annex A.1(2).

NOTE A more accurate effective cross section for outstand compression elements may be taken from Annex C of EN 1993-1-3.

(3) For flange elements of I-sections and box girders the stress ratio ψ used in Table 4.1 or Table 4.2 should be based on the properties of the gross cross-sectional area, due allowance being made for shear lag in the flanges if relevant. For web elements the stress ratio ψ used in Table 4.1 should be obtained using a stress distribution obtained with the effective area of the compression flange and the gross area of the web.

NOTE If the stress distribution comes from different stages of construction (as e.g. in a composite bridge) the stresses from the various stages may first be calculated with a cross section consisting of effective flanges and gross web and added. This stress distribution determines an effective web section that can be used for all stages to calculate the final stress distribution.

(4) Except as given in 4.4(5), the plate slenderness $\overline{\lambda}_p$ of an element may be replaced by:

$$\overline{\lambda}_{p,red} = \overline{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_y / \gamma_{M0}}} \qquad (4.4)$$

where $\sigma_{com,Ed}$ is the maximum design compressive stress in the element determined using the effective[p] area of the section caused by all simultaneous actions.

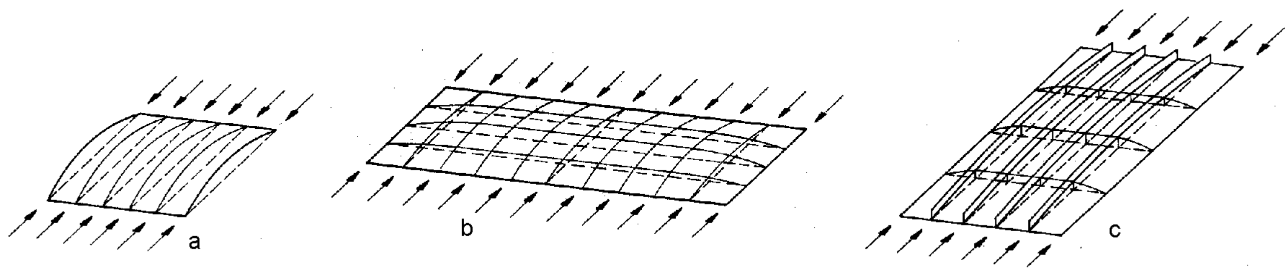
NOTE 1 The above procedure is conservative and requires an iterative calculation in which the stress ratio ψ (see Table 4.1 and Table 4.2) is determined at each step from the stresses calculated on the effective[p] cross-section defined at the end of the previous step.

NOTE 2 See also alternative procedure in 5.5.2 of EN 1993-1-3.

(5) For the verification of the design buckling resistance of a class 4 member using 6.3.1, 6.3.2 or 6.3.4 of EN 1993-1-1, either the plate slenderness $\overline{\lambda}_p$ should be used or $\overline{\lambda}_{p,red}$ with $\sigma_{com,Ed}$ based on second order analysis with global imperfections.

(6) For aspect ratios $a/b < 1$ a column type of buckling may be relevant and the check should be performed according to 4.5.3 using the reduction factor ρ_c.

NOTE This applies e.g. for flat elements between transverse stiffeners where plate buckling could be column-like and require a reduction factor ρ_c close to χ_c as for column buckling, see Figure 4.3.



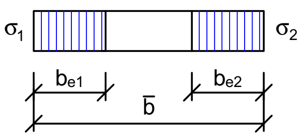
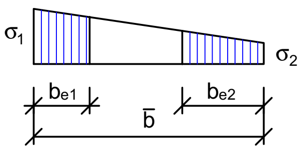
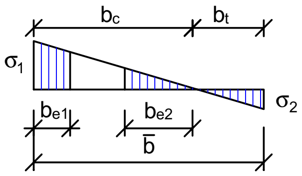
a) column-like behaviour of plates without longitudinal supports

b) column-like behaviour of an unstiffened plate with a small aspect ratio α

c) column-like behaviour of a longitudinally stiffened plate with a large aspect ratio α

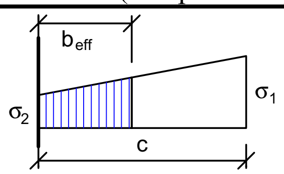
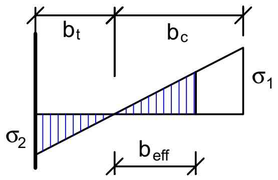
Figure 4.3: Column-like behaviour

Table 4.1: Internal compression elements

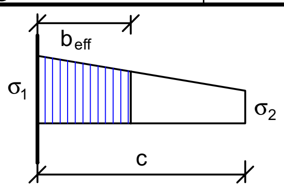
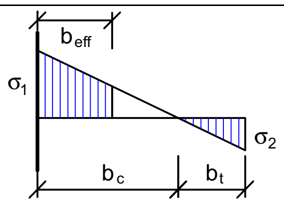
Stress distribution (compression positive)	Effective[p] width b_{eff}
	$\psi = 1$: $b_{eff} = \rho \bar{b}$ $b_{e1} = 0,5\, b_{eff}$ $b_{e2} = 0,5\, b_{eff}$
	$1 > \psi \quad 0$: $b_{eff} = \rho \bar{b}$ $b_{e1} = \frac{2}{5-\psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho b_c = \rho \bar{b} / (1-\psi)$ $b_{e1} = 0,4\, b_{eff}$ $b_{e2} = 0,6\, b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	$-1 > \psi > -3$
Buckling factor k_σ	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	23,9	$5,98 (1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$1 > \psi \quad 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_c = \rho c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \quad \psi \quad -3$
Buckling factor k_σ	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$

Stress distribution (compression positive)	Effective[p] width b_{eff}
	$1 > \psi \quad 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_c = \rho c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0,578 / (\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,1\psi^2$	23,8

4.5 Plate elements with longitudinal stiffeners

4.5.1 General

(1) For plate elements with longitudinal stiffeners the effective[p] areas from local buckling of the various subpanels between the stiffeners and the effective[p] areas from the global buckling of the stiffened panel shall be accounted for.

(2) The effective[p] section area of each subpanel should be determined by a reduction factor in accordance with 4.4 to account for local plate buckling. The stiffened plate with effective[p] section areas for the stiffeners should be checked for global plate buckling (e.g. by modelling as an equivalent orthotropic plate) and a reduction factor ρ for overall plate buckling of the stiffened plate should be determined.

(3) The effective[p] section area of the compression zone of the stiffened plate should be taken as:

$$A_{c,eff} = \rho_c \; A_{c,eff,loc} + \sum b_{edge,eff} \, t \tag{4.5}$$

in which $A_{c,eff,loc}$ is composed of the effective[p] section areas of all the stiffeners and subpanels that are fully or partially in the compression zone except the effective parts supported by an adjacent plate element with the width $b_{edge,eff}$, see example in Figure 4.4.

(4) The area $A_{c,eff,loc}$ should be obtained from:

$$A_{c,eff,loc} = A_{s\ell,eff} + \sum_c \rho_{loc} \; b_{c,loc} \; t \tag{4.6}$$

where $\sum_c$ applies to the part of the stiffened panel width that is in compression except the parts $b_{edge,eff}$, see Figure 4.4

$A_{s\ell,eff}$ is the sum of the effective[p] section according to 4.4 of all longitudinal stiffeners with gross area $A_{s\ell}$ located in the compression zone

$b_{c,loc}$ is the width of the compressed part of each subpanel

ρ_{loc} is the reduction factor from 4.4(2) for each subpanel.

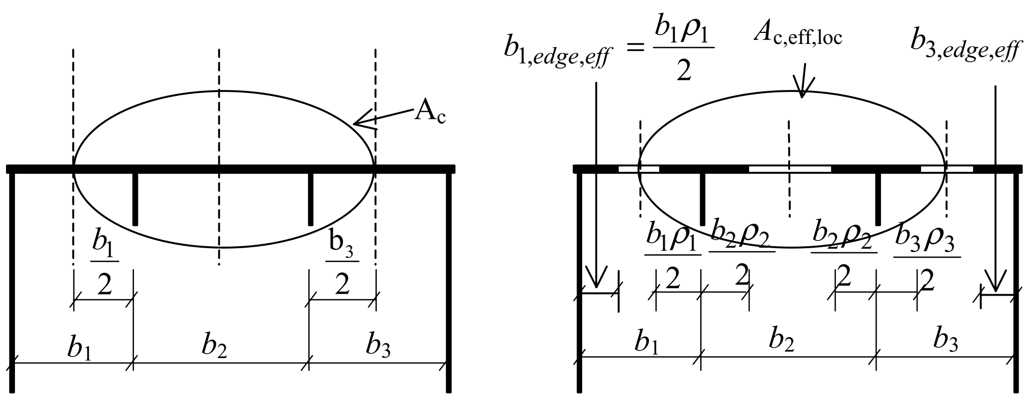


Figure 4.4: Definition of gross area A_c and and effective Area $A_{c,eff,loc}$ for stiffened plates under uniform compression (for non-uniform compression see Figure A.1)

NOTE For non-uniform compression see Figure A.1.

(5) In determining the reduction factor ρ_c for overall buckling the possibility of occurrence of column-type buckling, which requires a more severe reduction factor than for plate buckling, should be accounted for.

(6) This may be performed by interpolation in accordance with 4.5.4(1) between a reduction factor ρ for plate buckling and a reduction factor χ_c for column buckling to determine ρ_c.

(7) The reduction of the compressed area $A_{c,eff,loc}$ through ρ_c may be taken as a uniform reduction across the whole cross section.

(8) If shear lag is relevant (see 3.3), the effective cross-sectional area $A_{c,eff}$ of the compression zone of the stiffened plate element should then be taken as $A^*_{c,eff}$ accounting not only for local plate buckling effects but also for shear lag effects.

(9) The effective cross-sectional area of the tension zone of the stiffened plate element should be taken as the gross area of the tension zone reduced for shear lag if relevant, see 3.3.

(10) The effective section modulus W_{eff} should be taken as the second moment of area of the effective cross section divided by the distance from its centroid to the mid depth of the flange plate.

4.5.2 Plate type behaviour

(1) The relative plate slenderness $\bar{\lambda}_p$ of the equivalent plate is defined as:

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cr,p}}} \tag{4.7}$$

with $\beta_{A,c} = \dfrac{A_{c,eff,loc}}{A_c}$

where A_c is the gross area of the compression zone of the stiffened plate except the parts of subpanels supported by an adjacent plate element, see Figure 4.4 (to be multiplied by the shear lag factor if shear lag is relevant, see 3.3)

$A_{c,eff,loc}$ is the effective[p] area of the same part of the plate with due allowance made for possible plate buckling of subpanels and/or of stiffened plate elements

(2) The reduction factor ρ for the equivalent orthotropic plate is obtained from 4.4(2) provided $\bar{\lambda}_p$ is calculated from equation (4.5).

NOTE For calculation of $\sigma_{cr,p}$ see Annex A.

4.5.3 Column type buckling behaviour

(1) The elastic critical column buckling stress $\sigma_{cr,c}$ of an unstiffened (see 4.4) or stiffened (see 4.5) plate should be taken as the buckling stress of the unstiffened or stiffened plate with the supports along the longitudinal edges removed.

(2) For an unstiffened plate the elastic critical column buckling stress $\sigma_{cr,c}$ of an unstiffened plate may be obtained from

$$\sigma_{cr,c} = \frac{\pi^2 E t^2}{12\left(1-\nu^2\right)a^2} \tag{4.8}$$

(3) For a stiffened plate $\sigma_{cr,c}$ may be determined from the elastic critical column buckling stress $\sigma_{cr,st}$ of the stiffener closest to the panel edge with the highest compressive stress as follows:

$$\sigma_{cr,st} = \frac{\pi^2 E I_{sl,1}}{A_{sl,1}\, a^2} \tag{4.9}$$

where $I_{sl,1}$ is the second moment of area of the stiffener, relative to the out-of-plane bending of the plate,

A_{sl1} is the gross cross-sectional area of the stiffener and the adjacent parts of the plate according to Figure A.1

NOTE $\sigma_{cr,c}$ may be obtained from $\sigma_{cr,c} = \sigma_{cr,st} \frac{b_c}{\bar{b}}$ where $\sigma_{cr,c}$ is related to the compressed edge of the plate, and $\bar{b}, b_c$ are geometric values from the stress distribution used for the extrapolation, see Figure A.1.

(4) The relative column slenderness $\bar{\lambda}_c$ is defined as follows:

$$\bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr,c}}} \quad \text{for unstiffened plates} \tag{4.10}$$

$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c}\, f_y}{\sigma_{cr,c}}} \quad \text{for stiffened plates} \tag{4.11}$$

with $\beta_{A,c} = \frac{A_{s\ell,1,eff}}{A_{s\ell,1}}$

$A_{s\ell,1}$ is defined in 4.5.3(3) and

$A_{s\ell,1,eff}$ is the effective cross-sectional area of the stiffener with due allowance for plate buckling, see Figure A.1

(5) The reduction factor χ_c should be obtained from 6.3.1.2 of EN 1993-1-1. For unstiffened plates $\alpha = 0{,}21$ corresponding to buckling curve a should be used. For stiffened plates α should be magnified to account for larger initial imperfection in welded structures and replaced by α_e:

$$\alpha_e = \alpha + \frac{0{,}09}{i/e} \tag{4.12}$$

with $i = \sqrt{\frac{I_{st}}{A_{st}}}$

e = max (e_1, e_2) is the largest distance from the respective centroids of the plating and the one-sided stiffener (or of the centroids of either set of stiffeners when present on both sides) to the neutral axis of the column, see Figure A.1.

α = 0,34 (curve b) for closed section stiffeners

= 0,49 (curve c) for open section stiffeners

4.5.4 Interpolation between plate and column buckling

(1) The final reduction factor ρ_c should be obtained by interpolation between χ_c and ρ as follows:

$$\rho_c = (\rho - \chi_c)\,\xi\,(2 - \xi) + \chi_c \tag{4.13}$$

where $\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1$ but $0 \le \xi \le 1$

$\sigma_{cr,p}$ is the elastic critical plate buckling stress, see Annex A.1(2)

$\sigma_{cr,c}$ is the elastic critical column buckling stress according to 4.5.3(2) and (3), respectively.

4.6 Verification

(1) Member verification for direct stresses from compression and monoaxial bending should be performed as follows:

$$\eta_1 = \frac{N_{Ed}}{\frac{f_y\ A_{eff}}{\gamma_{M0}}} + \frac{M_{Ed} + N_{Ed}\ e_N}{\frac{f_y\ W_{eff}}{\gamma_{M0}}} \leq 1{,}0 \qquad (4.14)$$

where A_{eff} is the effective cross-section area in accordance with 4.3(3);

e_N is the shift in the position of neutral axis, see 4.3(3);

M_{Ed} is the design bending moment;

N_{Ed} is the design axial force;

W_{eff} is the effective elastic section modulus, see 4.3(4),

γ_{M0} is the partial factor, see application parts 2 to 6.

NOTE For compression and biaxial bending equation (4.14) may be extended to:

$$\eta_1 = \frac{N_{Ed}}{\frac{f_y\ A_{eff}}{\gamma_{M0}}} + \frac{M_{y,Ed} + N_{Ed}\ e_{y,N}}{\frac{f_y\ W_{y,eff}}{\gamma_{M0}}} + \frac{M_{z,Ed} + N_{Ed}\ e_{z,N}}{\frac{f_y\ W_{z,eff}}{\gamma_{M0}}} \leq 1{,}0 \qquad (4.15)$$

(2) Action effects M_{Ed} and N_{Ed} should include global second order effects where relevant.

(3) A stress gradient along the plate may be taken into account by the use of an effective length. As an alternative, the plate buckling verification of the panel may be carried out for the stress resultants at a distance 0,4a or 0,5b, whichever is the smallest, from the panel end where the stresses are the greater. In this case the gross sectional resistance needs to be checked at the end of the panel.

5 Resistance to shear

5.1 Basis

(1) This section gives rules for plate buckling effects from shear stresses at the ultimate limit state where the following criteria are met:

a) the panels are rectangular within the angle limit stated in 2.3,

b) stiffeners if any are provided in the longitudinal and/or transverse direction,

c) all holes and cut outs are small (see 2.3),

d) members are uniform.

(2) Plates with h_w/t greater than $\frac{72}{\eta}\varepsilon$ for an unstiffened web, or $\frac{31}{\eta}\varepsilon\sqrt{k_\tau}$ for a stiffened web, shall be checked for resistance to shear buckling and shall be provided with transverse stiffeners at the supports

NOTE 1 For h_w see Figure 5.1 and for k_τ see 5.3(3).

NOTE 2 The National Annex will define η. The value η = 1,20 is recommended. For steel grades higher than S460 η = 1,00 is recommended.

NOTE 3 Parameter $\varepsilon = \sqrt{\dfrac{235}{f_y\,[N/mm^2]}}$

5.2 Design resistance

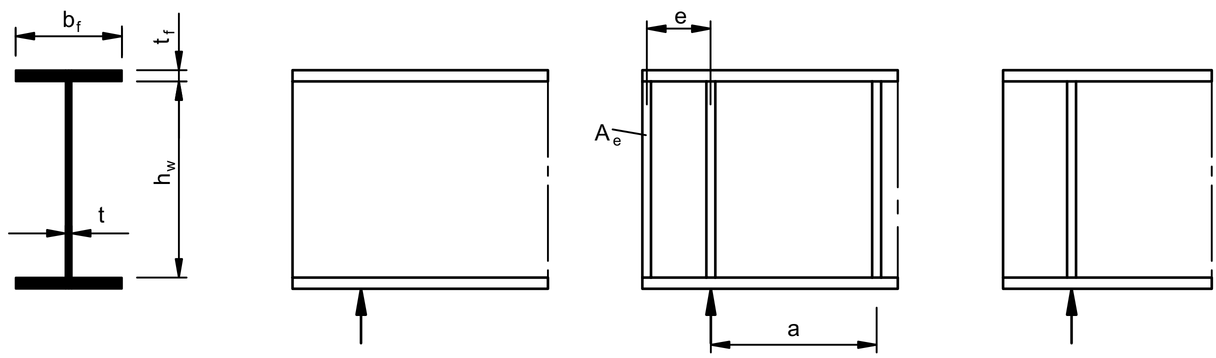
(1) For unstiffened or stiffened webs the design resistance for shear should be taken as:

$$V_{b,Rd} = \frac{\chi_V\, f_{yw}\, h_w\, t}{\sqrt{3}\,\gamma_{M1}} \tag{5.1}$$

$$\chi_V = \chi_w + \chi_f \quad \text{but not greater than } \eta. \tag{5.2}$$

in which χ_w is a factor for the contribution from the web and χ_f is a factor for the contribution from the flanges, determined according to 5.3 and 5.4, respectively.

(2) Stiffeners should comply with the requirements in 9.3 and welds should fulfil the requirement given in 9.3.5.



Cross section notations a) No end post b) Rigid end post c) Non-rigid end post

Figure 5.1: End-stiffeners

5.3 Contribution from webs

(1) For webs with transverse stiffeners at supports only and for webs with either intermediate transverse or longitudinal stiffeners or both, the factor χ_w for the contribution of the web to the shear buckling resistance should be obtained from Table 5.1 or Figure 5.2.

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

(2) Figure 5.1 shows various end supports for girders:

a) No end post, see 6.1 (2), type c);

b) Rigid end posts; this case is also applicable for panels at an intermediate support of a continuous girder, see 9.3.1;

c) Non rigid end posts, see 9.3.2.

(3) The slenderness parameter $\bar{\lambda}_w$ in Table 5.1 and Figure 5.2 may be taken as:

$$\bar{\lambda}_w = 0{,}76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} \tag{5.3}$$

where $\tau_{cr} = k_\tau \, \sigma_E$ (5.4)

NOTE Values for σ_E and k_τ may be taken from Annex A.

(4) For webs with transverse stiffeners at supports, the slenderness parameter $\bar{\lambda}_w$ may be taken as:

$$\bar{\lambda}_w = \frac{h_w}{86{,}4 \, t \, \varepsilon} \tag{5.5}$$

(5) For webs with transverse stiffeners at supports and with intermediate transverse or longitudinal stiffeners or both, the slenderness parameter $\bar{\lambda}_w$ may be taken as:

$$\bar{\lambda}_w = \frac{h_w}{37{,}4 \, t \, \varepsilon \sqrt{k_\tau}} \tag{5.6}$$

in which k_τ is the minimum shear buckling coefficient for the web panel.

When in addition to rigid stiffeners also non-rigid transverse stiffeners are used, the web panels between any two adjacent transverse stiffeners (e.g. $a_2 \times h_w$ and $a_3 \times h_w$) and web panels between adjacent rigid stiffeners containing non-rigid transverse stiffeners (e.g. $a_4 \times h_w$) should be checked for the smallest k_τ .

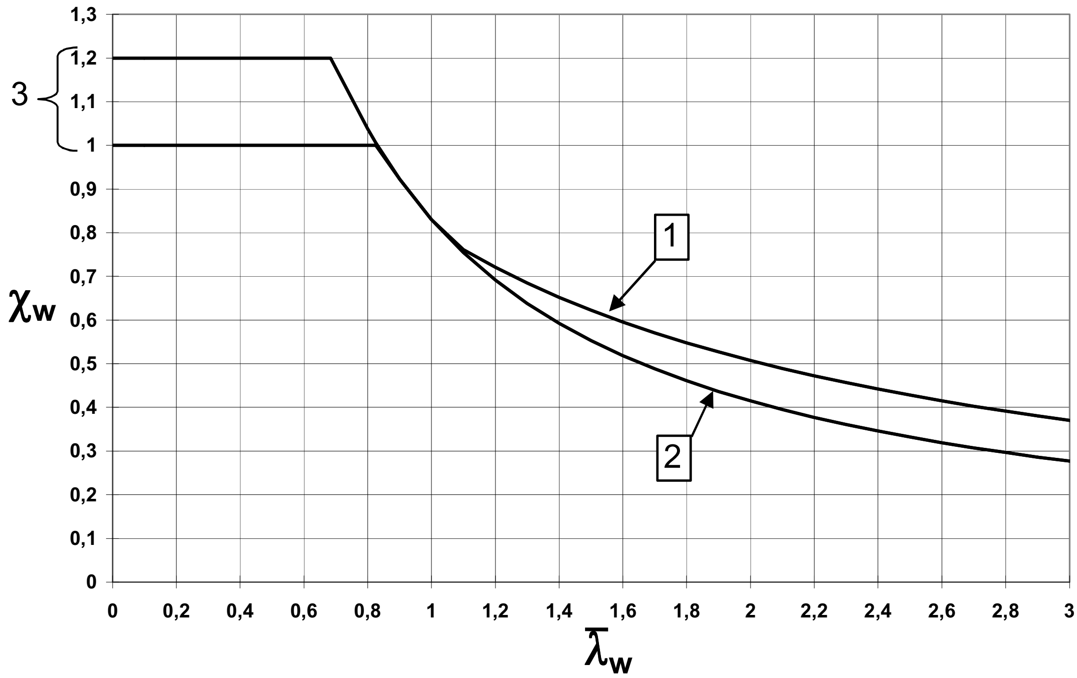
NOTE 1 Rigid boundaries may be assumed when flanges and transverse stiffeners are rigid, see 9.3.3. The web panels then are simply the panels between two adjacent transverse stiffeners (e.g. $a_1 \times h_{wi}$ in Figure 5.3).

NOTE 2 For non-rigid transverse stiffeners the minimum value k_τ may be taken from two checks:

1. check of two adjacent web panels with one flexible transverse stiffener
2. check of three adjacent web panels with two flexible transverse stiffeners

For procedure to determine k_τ see Annex A.3.

(6) The second moment of area of the longitudinal stiffeners should be reduced to 1/3 of their actual value when calculating k_τ. Formulae for k_τ taking this reduction into account in A.3 may be used.



1 Rigid end post
2 Non-rigid end post
3 Range of η

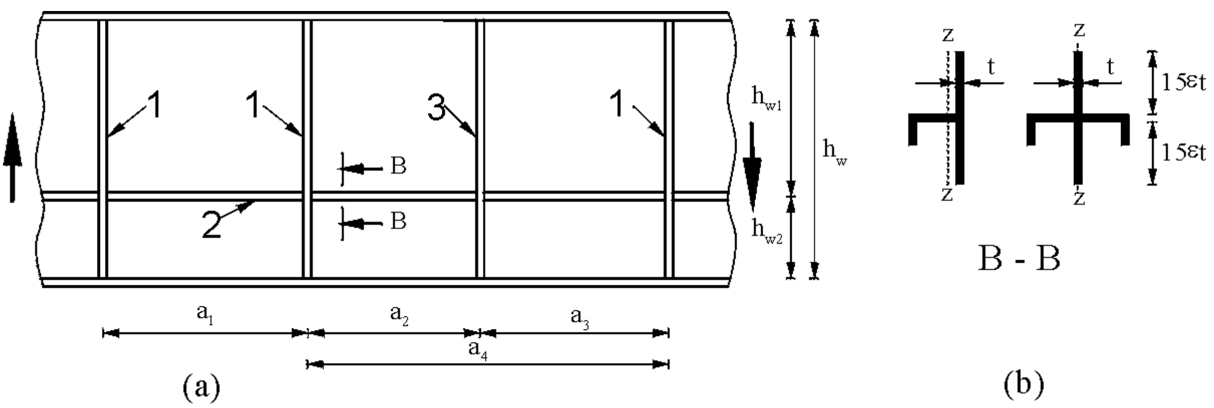
Figure 5.2: Shear buckling factor χ_w

(7) For webs with longitudinal stiffeners the slenderness parameter $\bar{\lambda}_w$ in (5) should not be taken as less than

$$\bar{\lambda}_w = \frac{h_{wi}}{37{,}4\, t\, \varepsilon \sqrt{k_{\tau i}}} \tag{5.7}$$

where h_{wi} and $k_{\tau i}$ refer to the subpanel with the largest slenderness parameter $\bar{\lambda}_w$ of all subpanels within the web panel under consideration.

NOTE To calculate $k_{\tau i}$ the expression given in A.3 may be used with $k_{\tau st} = 0$.



1 Rigid transverse stiffener
2 Longitudinal stiffener
3 Non-rigid transverse stiffener

Figure 5.3: Web with transverse and longitudinal stiffeners

5.4 Contribution from flanges

(1) If the flange resistance is not completely utilized in withstanding the bending moment ($M_{Ed} < M_{f,Rd}$) then a factor χ_f representing the contribution from the flanges may be included in the shear buckling resistance as follows:

$$\chi_f = \frac{b_f\ t_f^2\ f_{yf}\ \sqrt{3}}{c\ t\ h_w\ f_{yw}}\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{5.8}$$

in which b_f and t_f are taken for the flange leading to the lowest resistance,

b_f being taken as not larger than $15\varepsilon t_f$ on each side of the web,

$M_{f,Rd} = \frac{M_{f,k}}{\gamma_{M1}}$ is the design moment resistance of the cross section consisting of the effective flanges only,

$$c = a\left(0{,}25 + \frac{1{,}6\ b_f t_f^2\ f_{yf}}{t\ h_w^2\ f_{yw}}\right)$$

(2) When an axial force N_{Ed} is present, the value of $M_{f,Rd}$ should be reduced by a factor:

$$\left(1-\frac{N_{Ed}}{\frac{(A_{f1}+A_{f2})\ f_{yf}}{\gamma_{M1}}}\right) \tag{5.9}$$

where A_{f1} and A_{f2} are the areas of the top and bottom flanges.

5.5 Verification

(1) The verification should be performed as follows:

$$\eta_3 = \frac{V_{Ed}}{\chi_V h_w t\ f_{yw} / \left(\gamma_{M1}\sqrt{3}\right)} \leq 1{,}0 \tag{5.10}$$

where h_w is the clear distance between flanges;

t is the thickness of the plate;

V_{Ed} is the design shear force including shear from torque;

χ_v is the factor for shear resistance, see 5.2(1);

6 Resistance to transverse forces

6.1 Basis

(1) The resistance of the web of rolled beams and welded girders to transverse forces applied through a flange may be determined from the following rules, provided that the flanges are restrained in the lateral direction either by their own stiffness or by bracings.

(2) A load can be applied as follows:

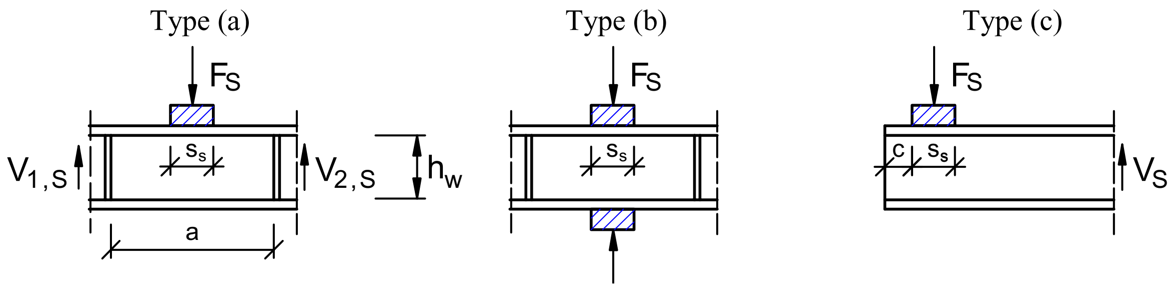
a) Load applied through one flange and resisted by shear forces in the web, see Figure 6.1 (a);

b) Load applied to one flange and transferred through the web directly to the other flange, see Figure 6.1 (b).

c) Load applied through one flange close to an unstiffened end, see Figure 6.1 (c)

(3) For box girders with inclined webs the resistance of both the web and flange should be checked. The internal forces to be taken into account are the components of the external load in the plane of the web and flange respectively.

(4) The interaction of the transverse force, bending moment and axial force should be verified using 7.2.



$$k_F = 6 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 3{,}5 + 2\left(\frac{h_w}{a}\right)^2 \qquad k_F = 2 + 6\left(\frac{s_s + c}{h_w}\right) \le 6$$

Figure 6.1: Buckling coefficients for different types of load application

6.2 Design resistance

(1) For unstiffened or stiffened webs the design resistance to local buckling under transverse forces should be taken as

$$F_{Rd} = \frac{f_{yw}\, L_{eff}\, t_w}{\gamma_{M1}} \tag{6.1}$$

where t_w is the thickness of the web

f_{yw} is the yield strength of the web

L_{eff} is the effective length for resistance to transverse forces, which should be determined from

$$L_{eff} = \chi_F\, \ell_y \tag{6.2}$$

where ℓ_y is the effective loaded length, see 6.5, appropriate to the length of stiff bearing s_s, see 6.3

χ_F is the reduction factor due to local buckling, see 6.4(1)

6.3 Length of stiff bearing

(1) The length of stiff bearing s_s on the flange is the distance over which the applied force is effectively distributed and it may be determined by dispersion of load through solid steel material at a slope of 1:1, see Figure 6.2. However, s_s should not be taken as larger than h_w.

(2) If several concentrated forces are closely spaced, the resistance should be checked for each individual force as well as for the sum of the forces with s_s as the centre-to-centre distance between the outer loads.

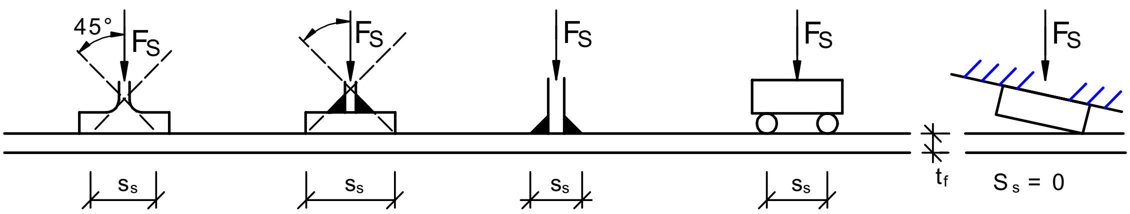


Figure 6.2: Length of stiff bearing

6.4 Reduction factor χ_F for effective length for resistance

(1) The reduction factor χ_F for effective length for resistance should be obtained from:

$$\chi_F = \frac{0,5}{\bar{\lambda}_F} \leq 1,0 \tag{6.3}$$

where $$\bar{\lambda}_F = \sqrt{\frac{\ell_y t_w f_{yw}}{F_{cr}}} \tag{6.4}$$

$$F_{cr} = 0,9\, k_F E \frac{t_w^3}{h_w} \tag{6.5}$$

(2) For webs without longitudinal stiffeners the factor k_F should be obtained from Figure 6.1.

NOTE 1 The values of k_F in Figure 6.1 are based on the assumption that the load is introduced by a device that prevents rotation of the flange.

NOTE 2 For webs with longitudinal stiffeners information may be given in the National Annex. The following rules are recommended:

For webs with longitudinal stiffeners k_F should be taken as

$$k_F = 6 + 2\left[\frac{h_w}{a}\right]^2 + \left[5,44\frac{b_1}{a} - 0,21\right]\sqrt{\gamma_s} \tag{6.6}$$

where b_1 is the depth of the loaded subpanel taken as the clear distance between the loaded flange and the stiffener

$$\gamma_s = 10,9\frac{I_{s\ell 1}}{h_w t_w^3} \leq 13\left[\frac{a}{h_w}\right]^3 + 210\left[0,3 - \frac{b_1}{h_w}\right] \tag{6.7}$$

where $I_{s\ell 1}$ is the second moments of area of the stiffener closest to the loaded flange including contributing parts of the web according to Figure A.1.

Equation (6.6) is valid for $0,05 \leq \frac{b_1}{h_w} \leq 0,3$ and loading according to type a) in Figure 6.1.

(3) ℓ_y should be obtained from 6.5.

6.5 Effective loaded length

(1) The effective loaded length ℓ_y should be calculated using two dimensionless parameters m_1 and m_2 obtained from:

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w} \tag{6.8}$$

$$m_2 = 0,02\left(\frac{h_w}{t_f}\right)^2 \quad \text{if } \bar{\lambda}_F > 0,5$$
$$m_2 = 0 \quad \text{if } \bar{\lambda}_F \leq 0,5 \tag{6.9}$$

For box girders, b_f in equation (6.8) should be limited to $15\varepsilon t_f$ on each side of the web.

(2) For cases (a) and (b) in Figure 6.1, ℓ_y should be obtained using:

$$\ell_y = s_s + 2\,t_f \left(1 + \sqrt{m_1 + m_2}\right) \text{ , but } \ell_y \leq \text{ distance between adjacent transverse stiffeners} \tag{6.10}$$

(3) For case c) ℓ_y should be obtained as the smaller of the values obtained from the equations given in 6.5(2) and (3). However, s_s in 6.5(2) should be taken as zero if the structure that introduces the force does not follow the slope of the girder, see Figure 6.2.

$$\ell_y = \ell_e + t_f \sqrt{\frac{m_1}{2} + \left(\frac{\ell_e}{t_f}\right)^2 + m_2} \tag{6.11}$$

$$\ell_y = \ell_e + t_f \sqrt{m_1 + m_2} \tag{6.12}$$

$$\ell_e = \frac{k_F E\, t_w^2}{2\, f_{yw} h_w} \leq s_s + c \tag{6.13}$$

6.6 Verification

(1) The verification should be performed as follows:

$$\eta_2 = \frac{F_{Ed}}{\dfrac{f_{yw}\; L_{eff}\; t_w}{\gamma_{M1}}} \leq 1{,}0 \tag{6.14}$$

where F_{Ed} is the design transverse force;

L_{eff} is the effective length for resistance to transverse forces, see 6.2(2);

t_w is the thickness of the plate.

Compressive stresses are taken as positive.

7 Interaction

7.1 Interaction between shear force, bending moment and axial force

(1) Provided that η_3 (see 5.5) does not exceed 0,5 , the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If η_3 is more than 0,5 the combined effects of bending and shear in the web of an I or box girder should satisfy:

$$\eta_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\eta_3 - 1\right)^2 \leq 1{,}0 \tag{7.1}$$

where $M_{f,Rd}$ is the design plastic moment resistance of a section consisting only of the effective flanges;

$M_{pl,Rd}$ is the plastic resistance of the section (irrespective of section class).

For the above verification η_1 may be calculated using gross section properties. In addition section 4.6 and 5.5 should be fulfilled.

Action effects should include global second order effects of members where relevant.

NOTE Equation (7.1) applies also to class 1 and class 2 sections, see EN 1993-1-1. In this cases η_1 refer to plastic resistances.

(2) The criterion given in (1) should be verified at all sections other than those located at a distance less than $h_w/2$ from the interior support.

(3) The plastic moment of resistance $M_{f,Rd}$ of the cross-section consisting of the flanges only should be taken as the product of the design yield strength, the effective^p area of the flange with the smallest value of Af_y and the distance between the centroids of the flanges.

(4) If an axial force N_{Ed} is applied, then $M_{pl,Rd}$ should be replaced by the reduced plastic resistance moment $M_{N,Rd}$ according to 6.2.9 of EN 1993-1-1 and $M_{f,Rd}$ should be reduced according to 5.4(2). If the axial force is so large that the whole web is in compression 7.1(5) should be applied.

(5) A flange in a box girder should be verified using 7.1(1) taking $M_{f,Rd} = 0$ and τ_{Ed} as the average shear stress in the flange which should not be less than half the maximum shear stress in the flange. In addition the subpanels should be checked using the average shear stress within the subpanel and χ_w determined for shear buckling of the subpanel according to 5.3, assuming the longitudinal stiffeners to be rigid.

7.2 Interaction between transverse force, bending moment and axial force

(1) If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with bending and axial force, the resistance should be verified using 4.6, 6.6 and the following interaction expression:

$$\eta_2 + 0{,}8\,\eta_1 \leq 1{,}4 \tag{7.2}$$

(2) If the concentrated load is acting on the tension flange the resistance according to section 6 should be verified and in addition also 6.2.1(5) of EN 1993-1-1.

8 Flange induced buckling

(1) To prevent the possibility of the compression flange buckling in the plane of the web, the ratio h_w/t_w for the web should satisfy the following criterion:

$$\frac{h_w}{t_w} \leq k\,\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}} \tag{8.1}$$

where A_w is the cross area of the web

A_{fc} is the effective cross area of the compression flange

The value of the factor k should be taken as follows:

- plastic rotation utilized $k = 0{,}3$
- plastic moment resistance utilized $k = 0{,}4$
- elastic moment resistance utilized $k = 0{,}55$

(2) When the girder is curved in elevation, with the compression flange on the concave face, the ratio $\frac{h_w}{t_w}$ should satisfy the following criterion:

$$\frac{h_w}{t_w} \leq \frac{k\,\frac{E}{f_{yf}}\sqrt{\frac{A_w}{A_{fc}}}}{\sqrt{1+\frac{h_w E}{3\,r\,f_{yf}}}} \tag{8.2}$$

in which r is the radius of curvature of the compression flange.

NOTE The National Annex may give further information on flange induced buckling.

9 Stiffeners and detailing

9.1 General

(1) This section gives rules for components of plated structures in supplement to the plate buckling rules in sections 4 to 7.

(2) When checking buckling resistance, the section of a stiffener may be taken as the gross cross-sectional area of the stiffener plus a width of plate equal to $15\varepsilon t$ but not more than the actual dimension available, on each side of the stiffener avoiding any overlap of contributing parts to adjacent stiffeners, see Figure 9.1.

(3) In general the axial force in a transverse stiffener should be taken as the sum of the force resulting from shear (see 9.3.3(3)) and any concentrated load.

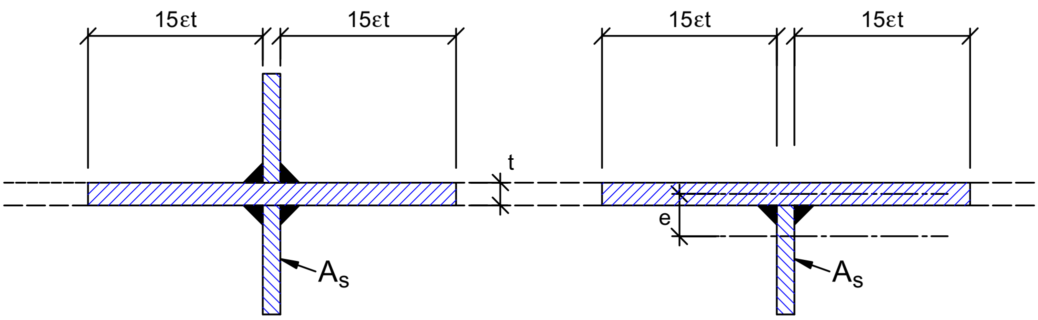


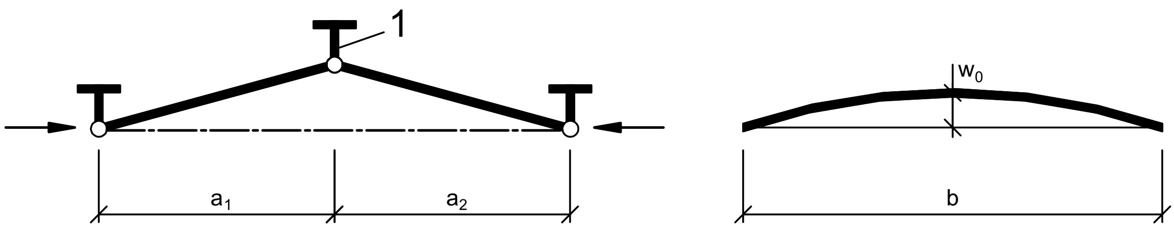
Figure 9.1: Effective cross-section of stiffener

9.2 Direct stresses

9.2.1 Minimum requirements for transverse stiffeners.

(1) In order to provide a rigid support for a plate with or without longitudinal stiffeners, intermediate transverse stiffeners should satisfy the minimum stiffness and strength requirements given below.

(2) The transverse stiffener should be treated as a simply supported beam with an initial sinusoidal imperfection w_0 equal to s/300, where s is the smallest of a_1, a_2 or b, see Figure 9.2 , where a_1 and a_2 are the lengths of the panels adjacent to the transverse stiffener under consideration and b is the depth or span of the transverse stiffener. Eccentricities should be accounted for.



1 Transverse stiffener

Figure 9.2: Transverse stiffener

(3) The transverse stiffener should carry the deviation forces from the adjacent compressed panels under the assumption that both adjacent transverse stiffeners are rigid and straight. The compressed panels and the longitudinal stiffeners are considered to be simply supported at the transverse stiffeners.

(4) It should be verified that based on a second order elastic analysis both the following criteria are satisfied:

- that the maximum stress in the stiffener under the design load should not exceed f_{yd}
- that the additional deflection should not exceed b/300

(5) In the absence of an axial force or/and transverse loads in the transverse stiffener both the criteria in (4) above may be assumed to be satisfied provided that the second moment of area I_{st} of the transverse stiffeners is not less than:

$$I_{st} = \frac{\sigma_m}{E}\left(\frac{b}{\pi}\right)^4 \left(1 + w_0 \frac{300}{b} u\right) \qquad (9.1)$$

with $\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \frac{N_{Ed}}{b}\left(\frac{1}{a_1} + \frac{1}{a_2}\right)$

$$u = \frac{\pi^2 \, E \, e_{max}}{\dfrac{f_y \; 300 \, b}{\gamma_{M1}}} \geq 1{,}0$$

where e_{max} is the distance from the extreme fibre of the stiffener to the centroid of the stiffener;

N_{Ed} is the largest design compressive force of the adjacent panels but not less than the largest compressive stress times half the effective[p] compression area of the panel including stiffeners;

$\sigma_{cr,c}$, $\sigma_{cr,p}$ are defined in 4.5.3 and Annex A.

NOTE Where out of plane loading is applied to the transverse stiffeners the simplification in (5) cannot be used.

(6) If the stiffener carries axial compression this should be increased with $\Delta N_{st} = \sigma_m b^2 / \pi^2$ in order to account for deviation forces. The criteria in (4) applies but ΔN_{st} need not to be considered when calculating the uniform stresses from axial load in the stiffener. Where the transverse stiffener is loaded by transverse force or transverse and axial force the requirement of (4) may be verified under the assumption of a class 3 section taking account of the following additional uniformly distributed lateral load q acting on the length b:

$$q = \frac{\pi}{4}\sigma_m \left(w_0 + w_{el}\right) \qquad (9.2)$$

where σ_m is defined in (5) above

w_0 is defined in Figure 9.2

w_{el} is the elastic deformation, that may be either determined iteratively or be taken as the maximum additional deflection b/300

(7) Unless are more sophisticated analysis is carried out in order to avoid torsional buckling of stiffeners with open cross-sections with only small warping resistance, the following criterion should be satisfied:

$$\frac{I_T}{I_p} \geq 5{,}3 \frac{f_y}{E} \qquad (9.3)$$

where I_p is the polar second moment of area of the stiffener alone around the edge fixed to the plate;

I_T is the St. Venant torsional constant for the stiffener alone.

(8) Stiffeners with warping stiffness should either fulfil (7) or the criterion

$$\sigma_{cr} \geq \theta\, f_y \qquad (9.4)$$

where σ_{cr} is the critical stress for torsional buckling not considering rotational restraint from the plate;

θ is a parameter to ensure class 3 behaviour.

NOTE The parameter θ may be given in the National Annex. The value θ = 6 is recommended.

9.2.2 Minimum requirements for longitudinal stiffeners

(1) The requirements concerning torsional buckling in 9.2.1(7) and (8) also applies to longitudinal stiffeners.

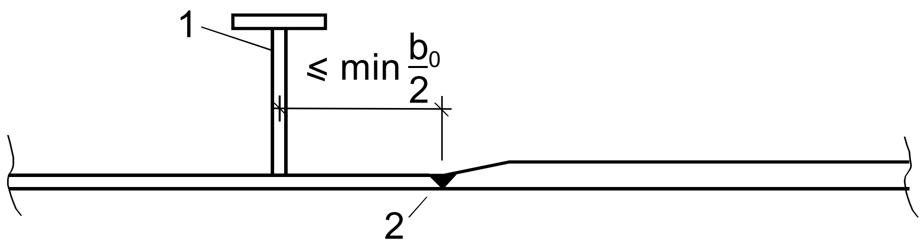
(2) Discontinuous longitudinal stiffeners that do not pass through openings made in the transverse stiffeners or are not connected to either side of the transverse stiffeners should be:

– used only for webs (i.e. not allowed in flanges)
– neglected in global analysis
– neglected in the calculation of stresses
– considered in the calculation of the effective[p] widths of web sub-panels
– considered in the calculation of the critical stresses.

(3) Strength assessments for stiffeners may be performed according to 4.5.3 and 4.6.

9.2.3 Splices of plates

(1) Welded transverse splices of plates with changes in plate thickness should be at the transverse stiffener, see Figure 9.3. The effects of eccentricity need not be taken into account where the distance to the stiffener stiffening the plate with the smaller thickness does not exceed $\min\left(\frac{b_0}{2}\right)$, where b is the width of a single plate between longitudinal stiffeners.



1 Transverse stiffener
2 Transverse splice of plate

Figure 9.3: Splice of plates

9.2.4 Cut outs in stiffeners

(1) Cut outs in longitudinal stiffeners should not exceed the values given in Figure 9.4.

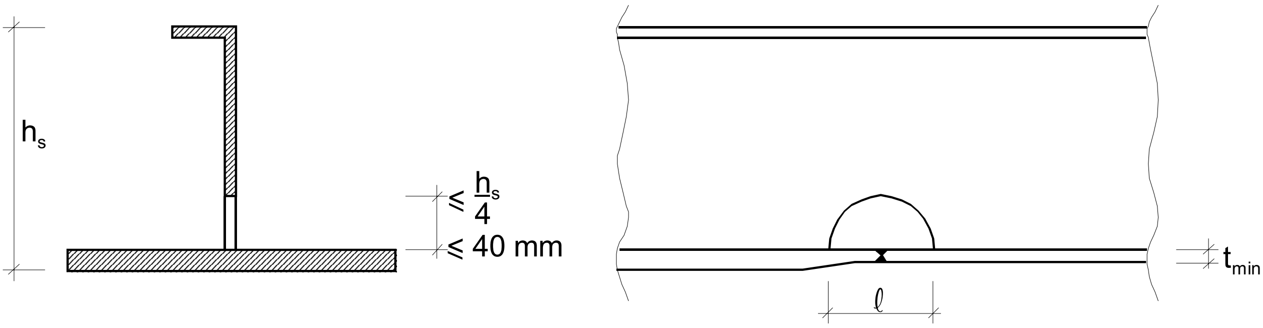


Figure 9.4: Cut outs for longitudinal stiffeners

(2) The maximum values ℓ are:

$\ell \leq 6\ t_{min}$ for flat stiffeners in compression

$\ell \leq 8\ t_{min}$ for other stiffeners in compression

$\ell \leq 15\ t_{min}$ for stiffeners without compression

where t_{min} is the lesser of the plate thicknesses

(3) The values ℓ in (2) for stiffeners in compression may be enhanced by $\sqrt{\frac{\sigma_{x,Rd}}{\sigma_{x,Ed}}}$ where $\sigma_{x,Ed} \leq \sigma_{x,Rd}$ unless $\ell = 15\,(\min t)$ is not exceeded.

(4) Cut outs in transverse stiffeners should not exceed the values given in Figure 9.5

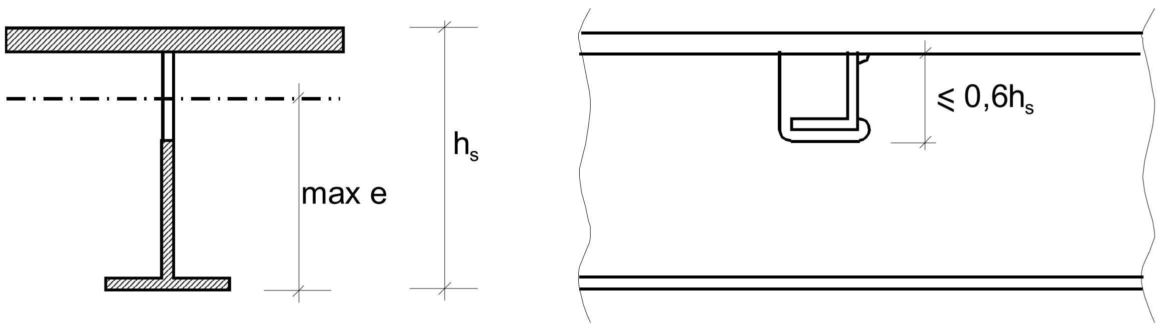


Figure 9.5: Cut outs for transverse stiffeners

(5) In addition to (4) the web should resist to the shear

$$V = \frac{I_{net}}{\max e}\frac{f_{yk}}{\gamma_{M0}}\frac{\pi}{b_G} \tag{9.5}$$

where I_{net} is the second moment of area for the net section

max e is the maximum distance from neutral axis of net section

b_G is the span of transverse stiffener

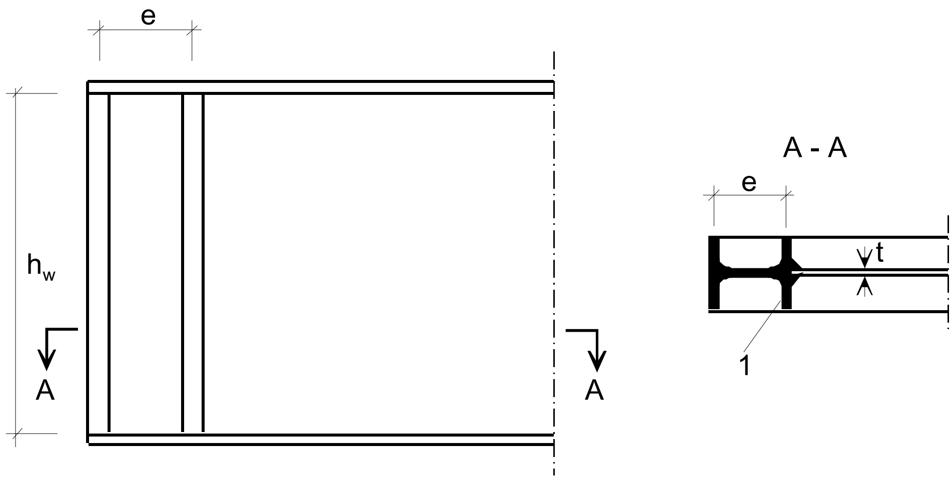
9.3 Shear

9.3.1 Rigid end post

(1) The rigid end post (see Figure 5.1) should act as a bearing stiffener resisting the reaction from bearings at the girder support (see 9.4), and as a short beam resisting the longitudinal membrane stresses in the plane of the web.

NOTE For the movements of bearing see EN 1993-2.

(2) A rigid end post may comprise two double-sided transverse stiffeners that form the flanges of a short beam of length h_w, see Figure 5.1 (b). The strip of web plate between the stiffeners forms the web of the short beam. Alternatively, an end post may be in the form of a rolled section, connected to the end of the web plate as shown in Figure 9.6.



1 Inserted section

Figure 9.6: Rolled section forming an end-post

(3) Each double sided stiffener consisting of flat plates should have a cross sectional area of at least $4h_w t^2 / e$, where e is the centre to centre distance between the stiffeners and $e > 0,1\, h_w$, see Figure 5.1 (b). Where the end-post is not made of flat stiffeners its section modulus should be at least $4h_w t^2$ for bending around a horizontal axis perpendicular to the web.

(4) As an alternative the girder end may be provided with a single double-sided stiffener and a vertical stiffener adjacent to the support so that the subpanel resists the maximum shear when designed with a non-rigid end post.

9.3.2 Stiffeners acting as non-rigid end post

(1) A non-rigid end post may be a single double sided stiffener as shown in Figure 5.1 (c). It may act as a bearing stiffener resisting the reaction at the girder support (see 9.4).

9.3.3 Intermediate transverse stiffeners

(1) Intermediate stiffeners that act as rigid supports to interior panels of the web should be checked for strength and stiffness.

(2) Other intermediate transverse stiffeners are considered to be flexible, their stiffness being considered in the calculation of k_τ in 5.3(5).

(3) The effective section of intermediate stiffeners acting as rigid supports for web panels should have a minimum second moment of area I_{st}:

$$\begin{aligned} &\text{if } a/h_w < \sqrt{2}: \quad I_{st} \geq 1{,}5\, h_w^3\, t^3 / a^2 \\ &\text{if } a/h_w \geq \sqrt{2}: \quad I_{st} \geq 0{,}75\, h_w\, t^3 \end{aligned} \tag{9.6}$$

The strength of intermediate rigid stiffeners should be checked for an axial force equal to $\left(V_{Ed} - \chi_w f_{yw} h_w t / \left(\sqrt{3}\, \gamma_{M1}\right)\right)$ according to 9.4, where χ_w is calculated for the web panel between adjacent transverse stiffeners assuming the stiffener under consideration is removed. In the case of variable shear forces the check is performed for the shear force at the distance $0{,}5h_w$ from the edge of the panel with the largest shear force.

9.3.4 Longitudinal stiffeners

(1) The strength should be checked for direct stresses if the stiffeners are taken into account for resisting direct stress.

9.3.5 Welds

(1) The web to flange welds may be designed for the nominal shear flow V_{Ed} / h_w if V_{Ed} does not exceed $\chi_w f_{yw} h_w t / \left(\sqrt{3}\, \gamma_{M1}\right)$. For larger values the weld between flanges and webs should be designed for the shear flow $\eta\, f_{yw} t / \left(\sqrt{3}\, \gamma_{M1}\right)$ unless the state of stress is investigated in detail.

(2) In all other cases welds should be designed to transfer forces between welds making up sections taking into account analysis method (elastic/plastic) and second order effects.

9.4 Transverse loads

(1) If the design resistance of an unstiffened web is insufficient, transverse stiffeners should be provided.

(2) The out-of-plane buckling resistance of the transverse stiffener under transverse load and shear force (see 9.3.3(3)) should be determined from 6.3.3 or 6.3.4 of EN 1993-1-1, using buckling curve c and a buckling length ℓ of not less than $0{,}75h_w$ where both ends are fixed laterally. A larger value of ℓ should be used for conditions that provide less end restraint. If the stiffeners have cut outs in the loaded end its cross sectional resistance should be checked at that end.

(3) Where single sided or other asymmetric stiffeners are used, the resulting eccentricity should be allowed for using 6.3.3 or 6.3.4 of EN 1993-1-1. If the stiffeners are assumed to provide lateral restraint to the compression flange they should comply with the stiffness and strength assumptions in the design for lateral torsional buckling.

10 Reduced stress method

(1) The following method may be used to determine stress limits for stiffened or unstiffened plates of a section to classify the section as a class 3 section.

NOTE 1 This method is an alternative to the effective width method specified in section 4 to 7. Shear lag effects should be taken into account where relevant.

NOTE 2 The National Annex may give limits of application for the methods.

(2) For unstiffened or stiffened panels subjected to combined stresses $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} class 3 section properties may be assumed, where

$$\frac{\rho\,\alpha_{ult,k}}{\gamma_{M1}} \geq 1 \qquad (10.1)$$

where $\alpha_{ult,k}$ is the minimum load amplifier for the design loads to reach the characteristic value of resistance of the most critical point of the plate, see (4)

ρ is the reduction factor depending on the plate slenderness $\overline{\lambda}_p$ to take account of plate buckling, see (5)

(3) The plate slenderness $\overline{\lambda}_p$ to determine ρ should be taken from

$$\overline{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}} \qquad (10.2)$$

where α_{cr} is the minimum load amplifier for the design loads to reach the elastic critical resistance of the plate under the complete stress field, see (6)

NOTE For calculating α_{cr} for the complete stress field stiffened plates may be modelled using the rules in section 4 and 5 however without reduction of the second moment of area of longitudinal stiffeners as specified in 5.3(6).

(4) In determining $\alpha_{ult,k}$ the yield criterion for plates of class 3-sections may be used for resistance:

$$\frac{1}{\alpha_{ult,k}^2} = \left(\frac{\sigma_{x,Ed}}{f_y}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_y}\right)\left(\frac{\sigma_{z,Ed}}{f_y}\right) + 3\left(\frac{\tau_{Ed}}{f_y}\right)^2 \qquad (10.3)$$

NOTE By using the equation (10.3) it is assumed that the resistance is reached when yielding occurs without plate buckling.

(5) The reduction factor ρ may be determined from either of the following methods:

a) the minimum value of the values

ρ_x for longitudinal stresses from 4.5.4(1) taking into account columnlike behaviour where relevant

ρ_z for transverse stresses from 4.5.4(1) taking into account columnlike behaviour where relevant

χ_v for shear stresses from 5.2(1)

each calculated for the slenderness $\bar{\lambda}_p$ according to equation (10.2)

NOTE This method leads to the verification formula:

$$\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)^2-\left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{f_y/\gamma_{M1}}\right)^2\leq\rho^2 \qquad (10.4)$$

NOTE For determining ρ_z for transverse stresses the rules in section 4 for direct stresses σ_x should be applied to σ_z in the z-direction. For consistency reasons section 6 should not be applied.

b) a value interpolated between the values ρ_x, ρ_z and χ_v as determined in a) by using the formula for $\alpha_{ult,k}$ as interpolation function

NOTE This method leads to the verification formate:

$$\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)^2-\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{\chi_v f_y/\gamma_{M1}}\right)^2\leq 1 \qquad (10.5)$$

NOTE The verification formulae (10.3), (10.4) and (10.5) include a platewise interaction between shear force, bending moment, axial force and transverse force, so that section 7 should not be applied.

(6) Where α_{cr} values for the complete stress field are not available and only $\alpha_{cr,i}$ values for the various components of the stress field $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} can be used, the α_{cr} value may be determined from:

$$\frac{1}{\alpha_{cr}}=\frac{1+\psi_x}{4\,\alpha_{cr,x}}+\frac{1+\psi_z}{4\,\alpha_{cr,z}}+\left[\left(\frac{1+\psi_x}{4\,\alpha_{cr,x}}+\frac{1+\psi_z}{4\,\alpha_{cr,z}}\right)^2+\frac{1-\psi_x}{2\,\alpha_{cr,x}^2}+\frac{1-\psi_z}{2\,\alpha_{cr,z}^2}+\frac{1}{\alpha_{cr,\tau}^2}\right]^{1/2} \qquad (10.6)$$

where $\alpha_{cr,x}=\dfrac{\sigma_{cr,x}}{\sigma_{x,Ed}}$

$\alpha_{cr,z}=\dfrac{\sigma_{cr,z}}{\sigma_{z,Ed}}$

$\alpha_{cr,\tau}=\dfrac{\tau_{cr,\tau}}{\tau_{\tau,Ed}}$

and $\sigma_{cr,x}$, $\sigma_{cr,z}$ τ_{cr}, ψ_x and ψ_z are determined from sections 4 to 6.

(7) Stiffeners and detailing of plate panels should be designed according to section 9.

Annex A [informative] – Calculation of reduction factors for stiffened plates

A.1 Equivalent orthotropic plate

(1) Plates with more than two longitudinal stiffeners may be treated as equivalent orthotropic plates.

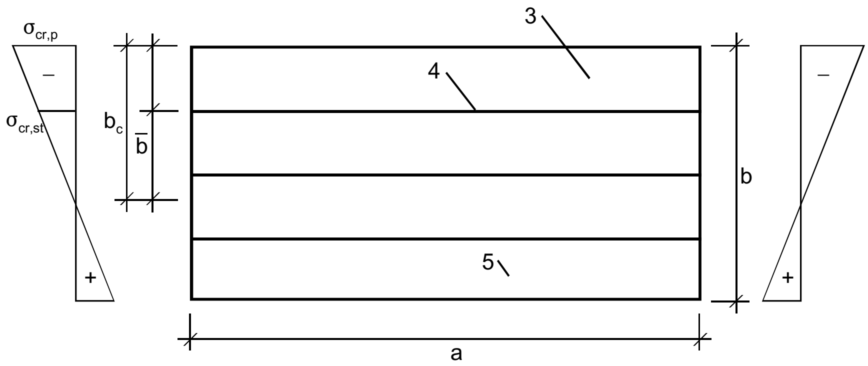
(2) The elastic critical plate buckling stress of the equivalent orthotropic plate is:

$$\sigma_{cr,p} = k_{\sigma,p}\ \sigma_E \qquad \text{(A.1)}$$

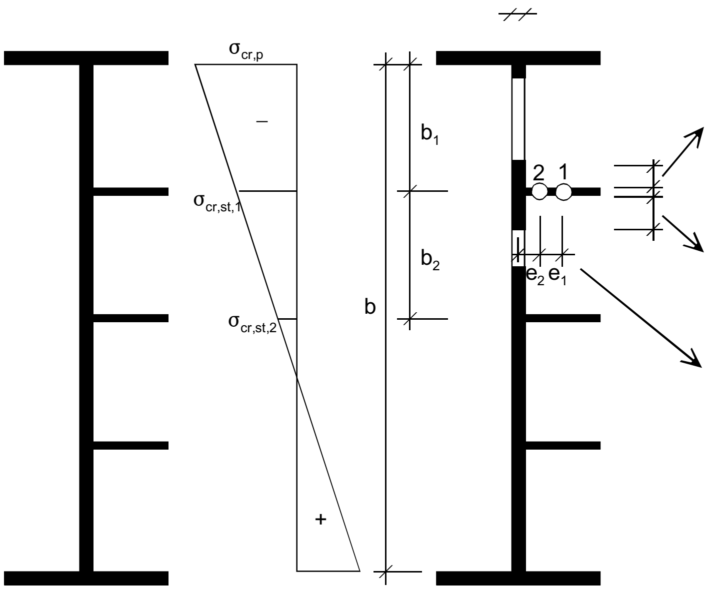
where $\sigma_E = \dfrac{\pi^2\ E\ t^2}{12\left(1-\nu^2\right)b^2} = 190000\left(\dfrac{t}{b}\right)^2$ in [MPa]

$k_{\sigma,p}$ is the buckling coefficient according to orthotropic plate theory with the stiffeners smeared over the plate

b, t are defined in Figure A.1



1 centroid of stiffeners
2 centroid of columns = stiffeners + cooperative plating
3 subpanel
4 stiffener
5 plate thickness t



gross area	effective area according to Table 4.1	
$\dfrac{3-\psi_1}{5-\psi_1}b_1$	$\dfrac{3-\psi_1}{5-\psi_1}b_{1,eff}$	$\psi_1 = \dfrac{\sigma_{cr,p}}{\sigma_{cr,st,1}}$
$\dfrac{2}{5-\psi_2}b_2$	$\dfrac{2}{5-\psi_2}b_{2,eff}$	$\psi_2 = \dfrac{\sigma_{cr,st,1}}{\sigma_{cr,st,2}} > 0$
$0{,}4\ b_2$	$0{,}4\ b_{2,eff}$	$\psi<0$

$e = \max(e_1\ ,\ e_2)$

Figure A.1: Notations for longitudinally stiffened plates

NOTE 1 The buckling coefficient $k_{\sigma,p}$ is obtained either from appropriate charts for smeared stiffeners or by relevant computer simulations; charts for discretely located stiffeners can alternatively be used provided local buckling in the subpanels can be ignored.

NOTE 2 $\sigma_{cr,p}$ is the elastic critical plate buckling stress at the edge of the panel where the maximum compression stress occurs, see Figure A.1.

NOTE 3 Where a web is of concern, the width b in equation (A.1) may be replaced by h_w.

NOTE 4 For stiffened plates with at least three equally spaced longitudinal stiffeners the plate buckling coefficient $k_{\sigma,p}$ (global buckling of the stiffened panel) may be approximated by

$$k_{\sigma,p} = \frac{2\left(\left(1+\alpha^2\right)^2 + \gamma - 1\right)}{\alpha^2(\psi+1)(1+\delta)} \quad \text{if} \quad \alpha \le \sqrt[4]{\gamma}$$
$$k_{\sigma,p} = \frac{4\left(1+\sqrt{\gamma}\right)}{(\psi+1)(1+\delta)} \quad \text{if} \quad \alpha > \sqrt[4]{\gamma} \qquad \text{(A.2)}$$

with: $\psi = \frac{\sigma_2}{\sigma_1} \ge 0{,}5$

$\gamma = \frac{\sum I_{sl}}{I_p}$

$\delta = \frac{\sum A_{sl}}{A_p}$

$\alpha = \frac{a}{b} \ge 0{,}5$

where: $\sum I_{sl}$ is the sum of the second moment of area of the whole stiffened plate;

I_p is the second moment of area for bending of the plate $= \frac{bt^3}{12\left(1-\upsilon^2\right)} = \frac{bt^3}{10{,}92}$;

$\sum A_{sl}$ is the sum of the gross area of the individual longitudinal stiffeners;

A_p is the gross area of the plate $= bt$;

σ_1 is the larger edge stress;

σ_2 is the smaller edge stress;

a, b and t are as defined in Figure A.1.

A.2 Critical plate buckling stress for plates with one or two stiffeners in the compression zone

A.2.1 General procedure

(1) If the stiffened plate has only one longitudinal stiffener in the compression zone the procedure in A.1 may be simplified by determining the elastic critical plate buckling stress $\sigma_{cr,p}$ in A.1(2) with the elastic critical stress for a isolated strut on an elastic foundation reflecting the plate effect in the direction perpendicular to this strut. The critical stress of the column may be obtained from A.2.2.

(2) For calculation of $A_{st,1}$ and $I_{st,1}$ the gross cross-section of the column should be taken as the gross area of the stiffener and adjacent parts of the plate defined as follows. If the subpanel is fully in compression, a portion $(3-\psi)/(5-\psi)$ of its width b_1 should be taken at the edge of the panel and $2/(5-\psi)$ at the edge with the highest stress. If the stresses change from compression to tension within the subpanel, a portion 0,4

of the width b_c of the compressed part of this subpanel should be taken as part of the column, see Figure A.2 and also Table 4.1. ψ is the stress ratio relative to the subpanel in consideration.

(3) The effective^p cross-sectional area $A_{st,1,eff}$ of the column should be taken as the effective^p cross-section of the stiffener and the adjacent effective^p parts of the plate, see Figure A.1. The slenderness of the plate elements in the column may be determined according to 4.4(4), with $\sigma_{com,Ed}$ calculated for the gross cross-section of the plate.

(4) If $\rho_c f_{yd}$,with ρ_c according to 4.5.4(1), is greater than the average stress in the column $\sigma_{com,Ed}$ no further reduction of the effective^p area of the column should be made. Otherwise the reduction according to equation (4.6) is replaced by:

$$A_{c,eff} = \frac{\rho_c \ f_y \ A_{st}}{\sigma_{com,Ed} \ \gamma_{M1}} \tag{A.3}$$

(5) The reduction mentioned in A.2.1(4) should be applied only to the area of the column. No reduction need be applied to other compressed parts of the plate, other than that for buckling of subpanels.

(6) As an alternative to using an effective^p area according to A.2.1(4), the resistance of the column can be determined from A.2.1(5) to (7) and checked to exceed the average stress $\sigma_{com,Ed}$. This approach can be used also in the case of multiple stiffeners in which the restraining effect from the plate may be neglected, that is the column is considered free to buckle out of the plane of the web.

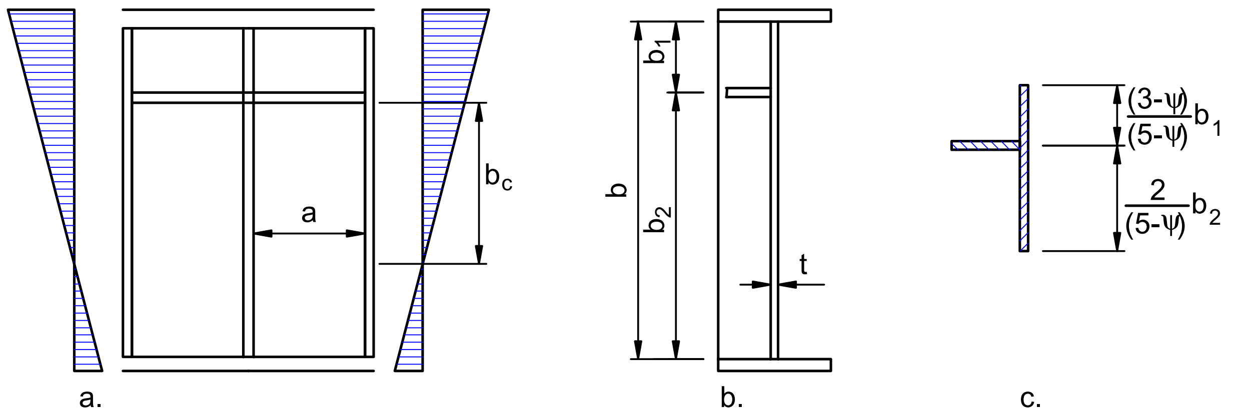


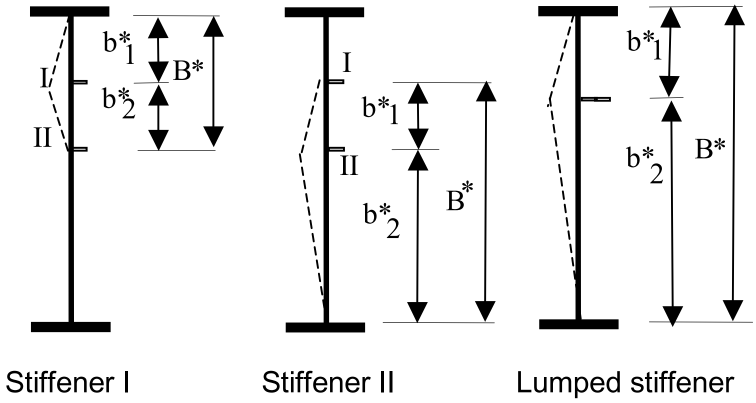
Figure A.2: Notations for plate with single stiffener in the compression zone

(7) If the stiffened plate has two longitudinal stiffeners in the compression zone, the one stiffener procedure described in A.2.1(1) can be applied, see Figure A.3. First, it is assumed that one of the stiffeners buckles while the other one acts a rigid support. Buckling of both stiffeners together is accounted for by considering a single lumped stiffener that is substituted for both individual ones such that:

a) its cross-sectional area and its second moment of area I_{st} are respectively the sum of that for the individual stiffeners

b) it is located at the location of the resultant of the respective forces in the individual stiffeners

For each of these situations illustrated in Figure A.3 a relevant value of $\sigma_{cr.p}$ is computed, see A.2.2(1), with $b_1=b_1^*$ and $b_2=b_2^*$ and $B^*=b_1^*+b_2^*$, see Figure A.3.



	Stiffener I	Stiffener II	Lumped stiffener
Cross-sectional area	$A_{st.1}$	$A_{st.2}$	$A_{st.1} + A_{st.2}$
Second moment of area	$I_{st,1}$	$I_{st,2}$	$I_{st,1} + I_{st,2}$

Figure A.3: Notations for plate with two stiffeners in the compression zone

A.2.2 Simplified model using a column restrained by the plate

(1) In the case of a stiffened plate with one longitudinal stiffener located in the compression zone, the elastic critical buckling stress of the stiffener can be calculated as follows ignoring stiffeners in the tension zone:

$$\sigma_{cr,st} = \frac{1{,}05\,E}{A_{st,1}} \frac{\sqrt{I_{st,1}\,t^3\,b}}{b_1\,b_2} \qquad \text{if } a \geq a_c$$

$$\sigma_{cr,st} = \frac{\pi^2\,E\,I_{st,1}}{A_{st,1}\,a^2} + \frac{E\,t^3\,b\,a^2}{4\,\pi^2\left(1-\nu^2\right)A_{st,1}\,b_1^2\,b_2^2} \qquad \text{if } a \leq a_c \qquad \text{(A.4)}$$

with $a_c = 4{,}33\sqrt[4]{\dfrac{I_{st,1}\,b_1^2\,b_2^2}{t^3\,b}}$

where $A_{st,1}$ is the gross area of the column obtained from A.2.1(2)

$I_{st,1}$ is the second moment of area of the gross cross-section of the column defined in A.2.1(2) about an axis through its centroid and parallel to the plane of the plate;

b_1,b_2 are the distances from longitudinal edges to the stiffener ($b_1+b_2 = b$).

NOTE For determining $\sigma_{cr,c}$ see NOTE 2 to 4.5.3(3).

(2) In the case of a stiffened plate with two longitudinal stiffeners located in the compression zone the elastic critical plate buckling stress is the lowest of those computed for the three cases using equation (A.4) with $b_1 = b_1^*$, $b_2 = b_2^*$ and $b = B^*$. The stiffeners in the tension zone are ignored in the calculation.

A.3 Shear buckling coefficients

(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient k_τ is:

$$\begin{aligned} k_\tau &= 5{,}34 + 4{,}00\,(h_w / a)^2 + k_{\tau st} \quad \text{when } a / h_w \geq 1 \\ k_\tau &= 4{,}00 + 5{,}34\,(h_w / a)^2 + k_{\tau st} \quad \text{when } a / h_w < 1 \end{aligned} \tag{A.5}$$

where $k_{\tau st} = 9 \left(\dfrac{h_w}{a} \right)^2 \sqrt[4]{\left(\dfrac{I_{sl}}{t^3\, h_w} \right)^3}$ but not less than $\dfrac{2{,}1}{t} \sqrt[3]{\dfrac{I_{sl}}{h_w}}$

a is the distance between transverse stiffeners (see Figure 5.3);

I_{sl} is the second moment of area of the longitudinal stiffener with regard to the z-axis, see Figure 5.3 (b). For webs with two or more longitudinal stiffeners, not necessarily equally spaced, I_{sl} is the sum of the stiffness of the individual stiffeners.

NOTE No intermediate non-rigid transverse stiffeners are allowed for in equation (A.5).

(2) The equation (A.5) also applies to plates with one or two longitudinal stiffeners, if the aspect ratio $\alpha = \dfrac{a}{h_w}$ satisfies $\alpha \geq 3$. For plates with one or two longitudinal stiffeners and an aspect ratio $\alpha < 3$ the shear buckling coefficient should be taken from:

$$k_\tau = 4{,}1 + \frac{6{,}3 + 0{,}18 \dfrac{I_{st}}{t^3 h_w}}{\alpha^2} + 2{,}2 \sqrt[3]{\frac{I_{st}}{t^3 h_w}} \tag{A.6}$$

Annex B [informative] – Non-uniform members

B.1 General

(1) For plated members, for which the regularity conditions of 4.1(1) do not apply, plate buckling may be verified by using the method in section 10.

NOTE The rules are applicable to webs of members with non parallel flanges (eg. haunched beams) and to webs with regular or irregular openings and non orthogonal stiffeners.

(2) For determining α_{ult} and α_{crit} FE-methods may be applied, see Annex C.

(3) The reduction factors ρ_x , ρ_z and χ_w may be obtained for $\bar{\lambda}_p$ from the appropriate plate buckling curve, see sections 4 and 5.

NOTE The reduction factors ρ_x, ρ_z and χ_w may also be determined from:

$$\rho = \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \bar{\lambda}_p}} \qquad \text{(B.1)}$$

where $\varphi_p = \frac{1}{2}\left(1 + \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right) + \bar{\lambda}_p\right)$

and $\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}}$

The values of $\bar{\lambda}_{p0}$ and α_p are in Table B.1. The values in Table B.1 have been calibrated to the buckling curves in sections 4 and 5. They give a direct relation to the equivalent geometric imperfection, by :

$$e_0 = \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right)\frac{t}{6}\,\frac{1 - \frac{\rho\bar{\lambda}_p}{\gamma_{M1}}}{1 - \rho\bar{\lambda}_p} \qquad \text{(B.2)}$$

Table B.1: Values for $\bar{\lambda}_{p0}$ and α_p

Product	predominant buckling mode	α_p	$\bar{\lambda}_{p0}$
hot rolled	direct stress for $\psi \geq 0$	0,13	0,70
	direct stress for $\psi < 0$ shear transverse stress		0,80
welded and cold formed	direct stress for $\psi \geq 0$	0,34	0,70
	direct stress for $\psi < 0$ shear transverse stress		0,80

B.2 Interaction of plate buckling and lateral torsional buckling of members

(1) The method given in B.1 may be extended to the verification of combined plate buckling and lateral torsional buckling of beams by calculating α_{ult} and α_{crit} as follows:

α_{ult} is the minimum load amplifier for the design loads to reach the characteristic value of resistance of the most critical cross section, neglecting any plate buckling or lateral torsional buckling

α_{cr} is the minimum load amplifier for the design loads to reach the elastic critical resistance of the beam including plate buckling and lateral torsional buckling modes

(2) In case α_{cr} contains lateral torsional buckling modes, the reduction factor ρ used should be the minimum of the reduction factor according to B.1(4) and the χ_{LT} – value for lateral torsional buckling according to 6.3.3 of EN 1993-1-1.

Annex C [informative] – FEM-calculations

C.1 General

(1) This Annex gives guidance for the use of FE-methods for ultimate limit state, serviceability limit state or fatigue verifications of plated structures.

NOTE 1 For FE-calculation of shell structures see EN 1993-1-6.

NOTE 2 This guidance applies to engineers experienced in the use of Finite Element methods.

(2) The choice of the FE-method depends on the problem to be analysed. The choice may be based on the following assumptions:

Table C.1: Assumptions for FE-methods

No	Material behaviour	Geometric behaviour	Imperfections, see section C.5	Example of use
1	linear	linear	no	elastic shear lag effect, elastic resistance
2	non linear	linear	no	plastic resistance in ULS
3	linear	non linear	no	critical plate buckling load
4	linear	non linear	yes	elastic plate buckling resistance
5	non linear	non linear	yes	elastic-plastic resistance in ULS

C.2 Use of FEM calculations

(1) In using FEM calculation for design special care should be given to

- the modelling of the structural component and its boundary conditions
- the choice of software and documentation
- the use of imperfections
- the modelling of material properties
- the modelling of loads
- the modelling of limit state criteria
- the partial factors to be applied

NOTE The National Annex may define the conditions for the use of FEM calculations in design.

C.3 Modelling for FE-calculations

(1) The choice of FE-models (shell models or volume models) and the meshing shall be in conformity with the required accuracy of results. In case of doubt the applicability of the mesh and the FE-size used should be verified by a sensivity check with successive refinement.

(2) The FE-modelling may be performed either for

- the component as a whole or
- a substructure as a part of the whole component,

NOTE An example for a component could be the web and/or the bottom plate of continuous box girders in the region of an inner support where the bottom plate is in compression. An example for a substructure could be a subpanel of a bottom plate under 2D loading.

(3) The boundary conditions for supports, interfaces and the details of load introduction should be chosen such that realistic or conservative results are obtained.

(4) Geometric properties should be taken as nominal.

(5) Where imperfections shall be provided they should be based on the shapes and amplitudes given in section C.5.

(6) Material properties should be based on the rules given in C.6(2).

C.4 Choice of software and documentation

(1) The software chosen shall be suitable for the task and be proven reliable.

NOTE Reliability can be proven by suitable bench mark tests.

(2) The meshing, loading, boundary conditions and other input data as well as the results shall be documented in a way that they can be checked or reproduced by third parties.

C.5 Use of imperfections

(1) Where imperfections need to be included in the FE-model these imperfections should include both geometric and structural imperfections.

(2) Unless a more refined analysis of the geometric imperfections and the structural imperfections is performed, equivalent geometric imperfections may be used.

NOTE 1 Geometric imperfections may be based on the shape of the critical plate buckling modes with amplitudes given in the National Annex. 80 % of the geometric fabrication tolerances is recommended.

NOTE 2 Structural imperfections in terms of residual stresses may be represented by a stress pattern from the fabrication process with amplitudes equivalent the mean (expected) values.

(3) The direction of the imperfection should be provided as appropriate for obtaining the lowest resistance.

(4) The assumptions for equivalent geometric imperfections according to Table C.2 and Figure C.1 may be used.

Table C.2: Equivalent geometric imperfections

type of imperfection	component	shape	magnitude
global	member with length ℓ	bow	see EN 1993-1-1, Table 5.1
global	longitudinal stiffener with length a	bow	min (a/400, b/400)
local	panel or subpanel with short span a or b	buckling shape	min (a/200, b/200)
local	stiffener subject to twist	bow twist	1 / 50

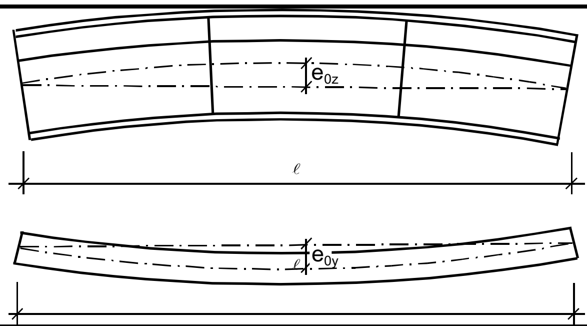
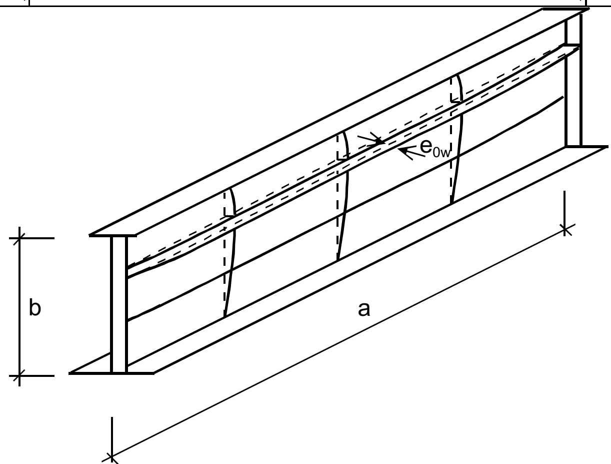
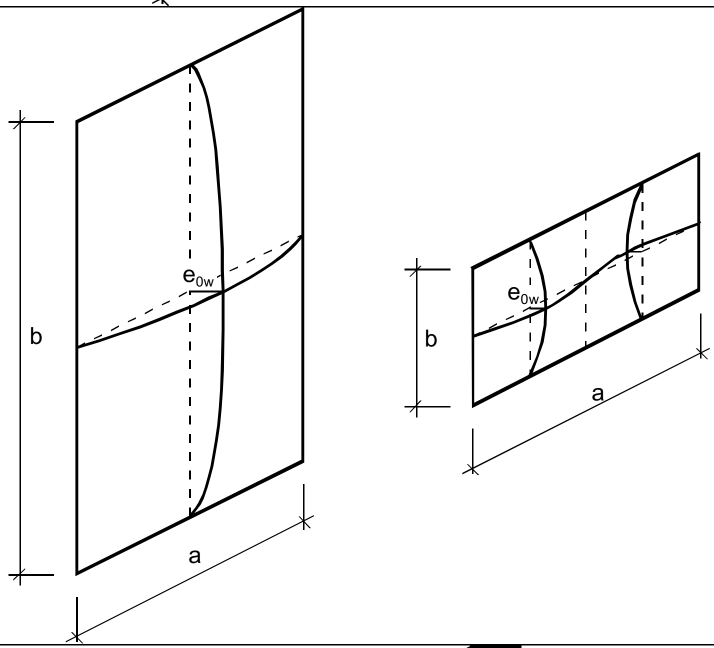
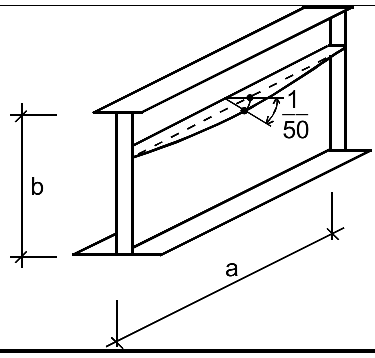
Type of imperfection	Component
global member with length ℓ	
global longitudinal stiffener with length a	
local panel or subpanel	
local stiffener or flange subject to twist	

Figure C.1: Modelling of equivalent geometric imperfections

(5) In combining these imperfections a leading imperfection should be chosen and the accompanying imperfections may be reduced to 70%.

NOTE 1 Any type of imperfection may be taken as the leading imperfection, the others may be taken as the accompanying.

NOTE 2 Equivalent geometric imperfections may be applied by substitutive disturbing forces.

C.6 Material properties

(1) Material properties should be taken as characteristic values.

(2) Depending on the accuracy required and the maximum strains attained the following approaches for the material behaviour may be used, see Figure C.2:

a) elastic-plastic without strain hardening

b) elastic-plastic with a pseudo strain hardening (for numerical reasons)

c) elastic-plastic with linear strain hardening

d) true stress-strain curve calculated from a technical stress-strain curve as measured as follows:

$$\sigma_{true} = \sigma\,(1+\varepsilon)$$
$$\varepsilon_{true} = \ell n\,(1+\varepsilon) \tag{C.1}$$

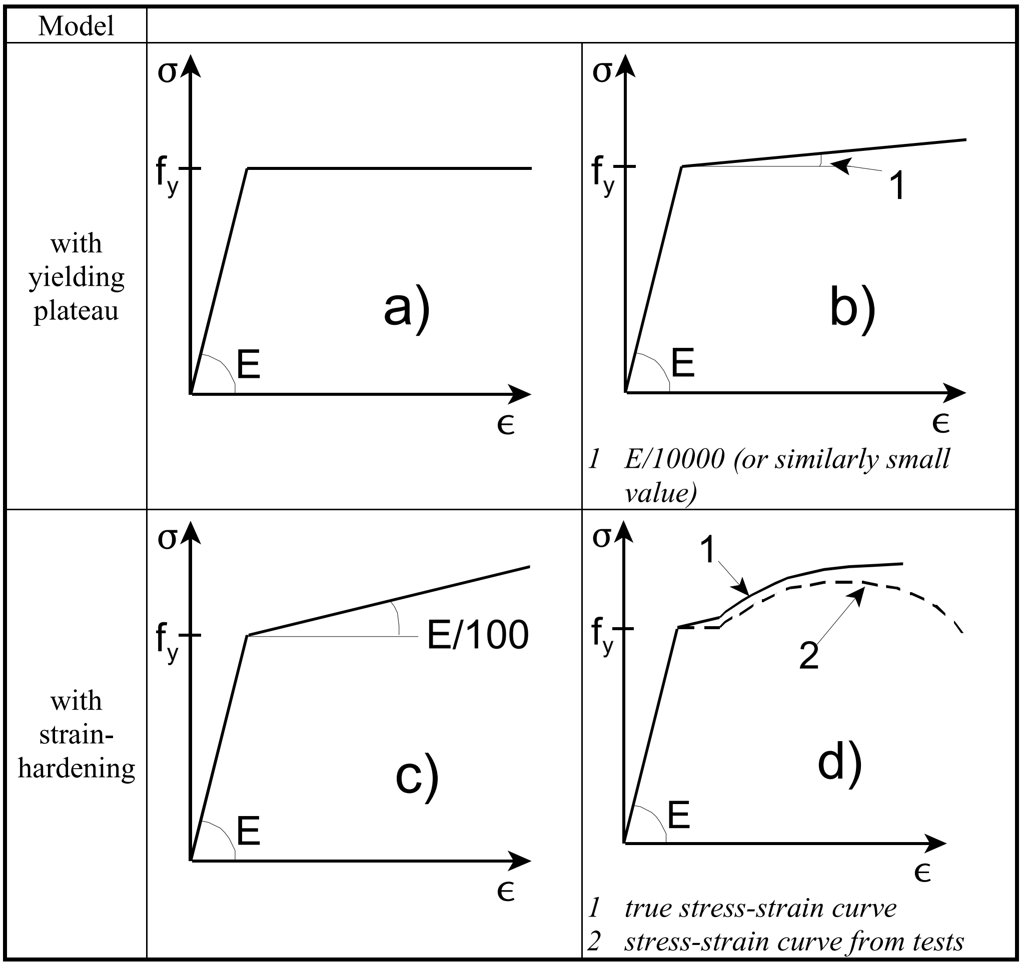


Figure C.2: Modelling of material behaviour

NOTE For the elastic modulus E the nominal value is relevant.

C.7 Loads

(1) The loads applied to the structures should include relevant load factors and load combination factors. For simplicity a single load multiplier α may be used.

C.8 Limit state criteria

(1) The following ultimate limit state criteria may be used:

1. for structures susceptible to buckling phenomena:

   attainment of the maximum load

2. for regions subjected to tensile stresses:

   attainment of a limit value of the principal membrane strain

NOTE 1 The National Annex may specify the limit of principal strain. A limit of 5% is recommended.

NOTE 2 As an alternative other criteria proceeding the limit state may be used: e.g. attainment of the yielding criterion or limitation of the yielding zone.

C.9 Partial factors

(1) The load magnification factor α_u to the ultimate limit state shall be sufficient to attain the required reliability.

(2) The magnification factor required for reliability should consist of two factors:

1. α_1 to cover the model uncertainty of the FE-modelling used
2. α_2 to cover the scatter of the loading and resistance models

(3) α_1 should be obtained from evaluations of tests calibrations, see Annex D to EN 1090.

(4) α_2 may be taken as γ_{M1} if instability governs and γ_{M2} if fracture governs.

(5) The verification should lead to

$$\alpha_u > \alpha_1 \, \alpha_2 \qquad \text{(C.2)}$$

NOTE The National Annex may give information on γ_{M1} and γ_{M2}. The use of γ_{M1} and γ_{M2} as specified in EN 1993-1-1 is recommended.

Annex D [informative] – Members with corrugated webs

D.1 General

(1) The rules given in this Annex D are valid for I-girders with trapezoidally or sinusoidally corrugated webs according to Figure D.1.

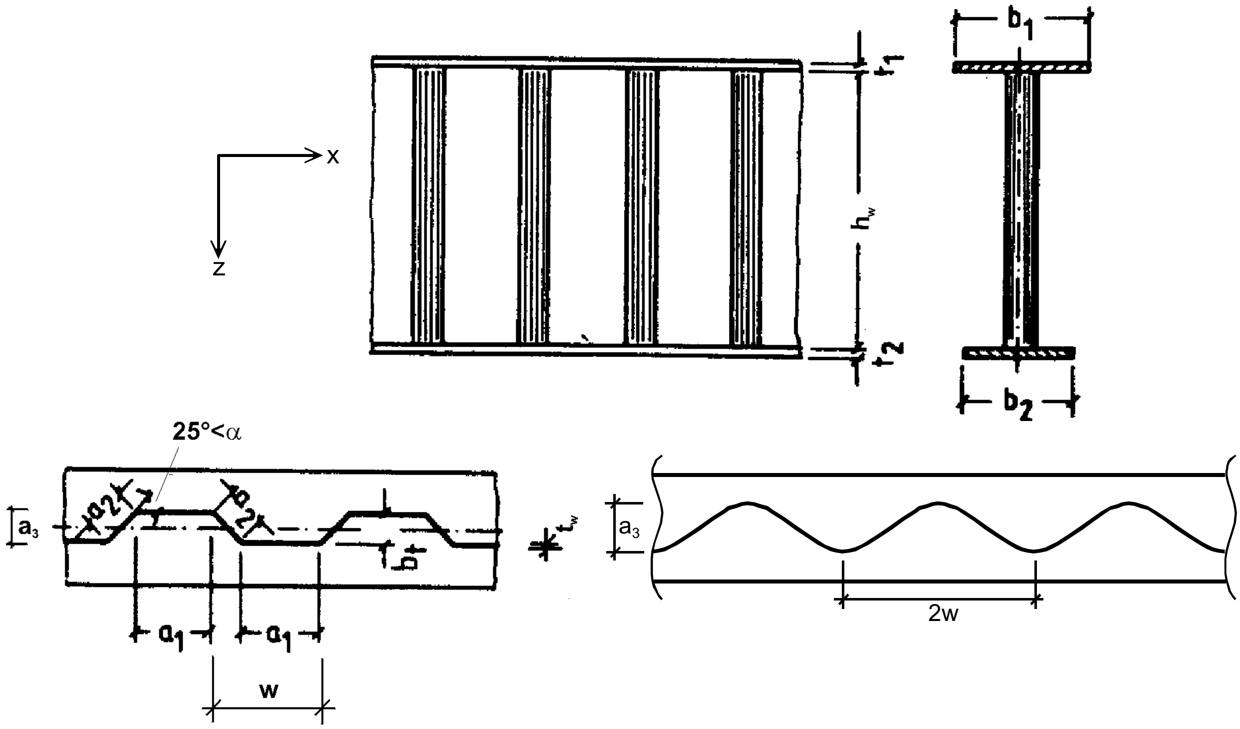


Figure D.1: Definitions

NOTE 1 Cut outs are not included in the rules for corrugated webs.

NOTE 2 For transverse loads the rules in 6 can be used as a conservative estimate.

D.2 Ultimate limit state

D.2.1 Bending moment resistance

(1) The bending moment resistance may be derived from:

$$M_{Rd} = \min \left\{ \underbrace{\frac{b_2 t_2 f_{y,r} h_w}{\gamma_{M0}}}_{\text{tension flange}} ; \underbrace{\frac{b_1 t_1 f_{y,r} h_w}{\gamma_{M0}}}_{\text{compression flange}} ; \underbrace{\frac{b_1 t_1 \chi f_y h_w}{\gamma_{M1}}}_{\text{compression flange}} \right\} \tag{D.1}$$

where $f_{y,r}$ includes the reduction due to transverse moments in the flanges

$f_{y,r} = f_y \, f_T$

$$f_T = 1 - 0{,}4 \sqrt{\frac{\sigma_x(M_z)}{\frac{f_y}{\gamma_{M0}}}}$$

M_z is the transverse moment in the flange

χ is the reduction force for lateral buckling according to 6.3 of EN 1993-1-1

NOTE 1 The transverse moment M_z may result from the shear flow introduction in the flanges as indicated in Figure D.2.

NOTE 2 For sinusoidally corrugated webs f_T is 1,0.

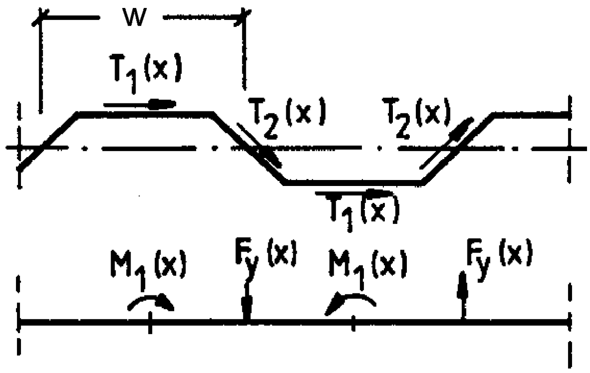


Figure D.2: Transverse moments M_z due to shear flow introduction into the flange

(2) The effective area of the compression flange should be determined according to 4.4(1) and (2) for the larger of the slenderness parameter $\bar{\lambda}_p$ defined in 4.4(2) with the following input:

a) $$k_\sigma = 0{,}43 + \left(\frac{b}{a}\right)^2 \tag{D.2}$$

where b is the largest outstand from weld to free edge

$a = a_1 + 2a_3$

b) $$k_\sigma = 0{,}55 \tag{D.3}$$

where $b = \frac{b_1}{2}$

D.2.2 Shear resistance

(1) The shear resistance V_{Rd} may be taken as:

$$V_{Rd} = \chi_c \frac{f_{yw}}{\gamma_{M1}\sqrt{3}} h_w t_w \tag{D.4}$$

where χ_c is the smallest of the reduction factors for local buckling $\chi_{c,\ell}$ and global buckling $\chi_{c,g}$ according to (2) and (3)

(2) The reduction factor $\chi_{c,\ell}$ for local buckling may be calculated from:

$$\chi_{c,\ell} = \frac{1{,}15}{0{,}9 + \bar{\lambda}_{c,\ell}} \leq 1{,}0 \tag{D.5}$$

The slenderness $\bar{\lambda}_{c,\ell}$ may be taken as

$$\bar{\lambda}_{c,\ell} = \sqrt{\frac{f_y}{\tau_{cr,\ell}\sqrt{3}}} \tag{D.6}$$

where the value $\tau_{cr,\ell}$ for local buckling of trapezoidally corrugated webs may be taken from

$$\tau_{cr,\ell} = 4{,}83\ E\left[\frac{t_w}{a_{max}}\right]^2 \qquad (D.7)$$

with $a_{max} = \max[a_1, a_2]$.

For sinusoidally corrugated webs $\tau_{cr,\ell}$ may be taken from

$$\tau_{cr,l} = \left(5{,}34 + \frac{a_3 s}{2h_w t_w}\right)\frac{\pi^2 E}{12(1-\nu^2)}\frac{2t_w}{s} \qquad (D.8)$$

(3) The reduction factor $\chi_{c,g}$ for global buckling should be taken as

$$\chi_{c,g} = \frac{1{,}5}{0{,}5 + \bar{\lambda}_{c,g}^2} \le 1{,}0 \qquad (D.9)$$

The slenderness $\bar{\lambda}_{c,g}$ may be taken as

$$\bar{\lambda}_{c,g} = \sqrt{\frac{f_y}{\tau_{cr,g}\sqrt{3}}} \qquad (D.10)$$

where the value $\tau_{cr,g}$ may be taken from

$$\tau_{cr,g} = \frac{32{,}4}{t_w\ h_w^2}\sqrt[4]{D_x\ D_z^3} \qquad (D.11)$$

where $D_x = \dfrac{E\ t^3}{12}\dfrac{w}{s}$

$D_z = \dfrac{E\ I_z}{w}$

w length of corrugation

s unfolded length

I_z second moment of area of one corrugation of length w, see Figure D.1

NOTE 1 s and I_z are determined from the actual shape of the corrugation.

NOTE 2 Equation (D.11) applied to plates with hinged edges.

D.2.3 Requirements for end stiffeners

(1) End stiffeners should be designed according to section 9.